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APPENDIX

1. Material properties

This appendix presents the properties of all materials (concrete, laminated veneer lumber or LVL, plywood and reinforcement) used in this project.

1.1 Concrete

Table A1-1 presents a summary of concrete used in the different experimental phases and its source.

Table A1-1. Type of concrete used in the different experimental phases and its source

Phase	Test	Concrete	Source
1	Short-term push-out	NWC, LSC	Laboratory
2	Short-term push-out	CLSC	Firth Concrete
3	Long-term push-out	CLSC	Firth Concrete
4	Short-term beam	NWC, CLSC SLSC	Firth Concrete Laboratory
5	Long-term beam	CLSC	Firth Concrete

Note: NWC for Normal Weight Concrete, LSC for Low Shrinkage Concrete, CLSC for Commercial Low Shrinkage Concrete, and SLSC for Special Low Shrinkage Concrete

Two types of 30 MPa concrete were used in Phase 1 short-term push-out test: low shrinkage concrete (LSC) for specimen G1 and normal weight concrete for all the other specimens. The concrete was prepared in the laboratory using mix design given in Table A1-2:

In the Phases 2, 3, 4, and 5, a commercially available low shrinkage concrete (CLSC) from Firth Concrete was used. The properties of this concrete are: 35 MPa, 650 microstrain, low shrinkage concrete with Eclipse admixture, 13 mm aggregate, and 120 mm slump.

Table A1-2. Mix design of concrete for the first phase of push-out test given in amount per cubic meter

30 MPa Normal Concrete (w/c = 0.60)		
GP cement	267	kg
Water	160	kg
13mm Greywacke stone	1000	kg
Christchurch Greywacke sand	900	kg
Water reducer	1	L
30 MPa Low Shrinkage Concrete		
GP cement	267	kg
Water	150	kg
19 mm limestone	1000	kg
Whangarei sand	900	kg
Super-plasticizer	1.5	L
Shrinkage reducing admixture	10	L

Table A1-3. Mix design of special low shrinkage concrete (SLSC) used in Phase 4 given in amount per cubic meter

Cement	200	kg
Water	160	L
19 mm limestone	950	kg
PAP7 crusher sand	550	kg
Natural sand	450	kg
Fly ash	100	kg
Super-plasticizer	2.5	L
Shrinkage reducing admixture	10	L

A special low shrinkage concrete (SLSC) was batched in the laboratory to mix design in Table A1-3 for one beam in Phase 4. The targeted compressive strength at 28 days was 35 MPa, and the expected drying shrinkage was 500 microstrain. Chemical admixtures used were Sika Viscocrete 5/500 (S/P) and Sika Control 40 (SRA). The fly ash was Gladstone fly ash from Australia and the cement was GP cement from Holcim Westport. Limestone was from Waiau quarry in North Canterbury and the natural sand was from Kaipara harbour in the North Island. PAP7 crusher sand was from Whangarei (PAP7 for premium all passing which means all sand were less than 7 mm).

Table A1-4. Phases 1, 2 and 3 (short- and long-term push-out tests) concrete tested mean properties

Concrete	Density (kg/m^3)	Compressive strength 28 day (MPa)	Elastic modulus (GPa)
LSC (Phase 1)	2328	42.7	29.1
CLSC (Phases 2 and 3)	2373	45.0	30.6

Table A1-5. Phase 4 (short-term beam tests) concrete tested properties

Beam	Concrete	Compressive strength (MPa)		Slump
		28 day	Test day	(mm)
A1, A2, B1, C2	CLSC G35	49.6	58.0	150
C1, D1, D2	CLSC G35	42.6	54.4	170
E1, G1	CLSC G35	41.5	48.2	190
F1, F2	CLSC G35	43.4	54.4	220
E2	NWC G25	25.4	31.0	200
B2	SLSC G35	28.0	38.8	100

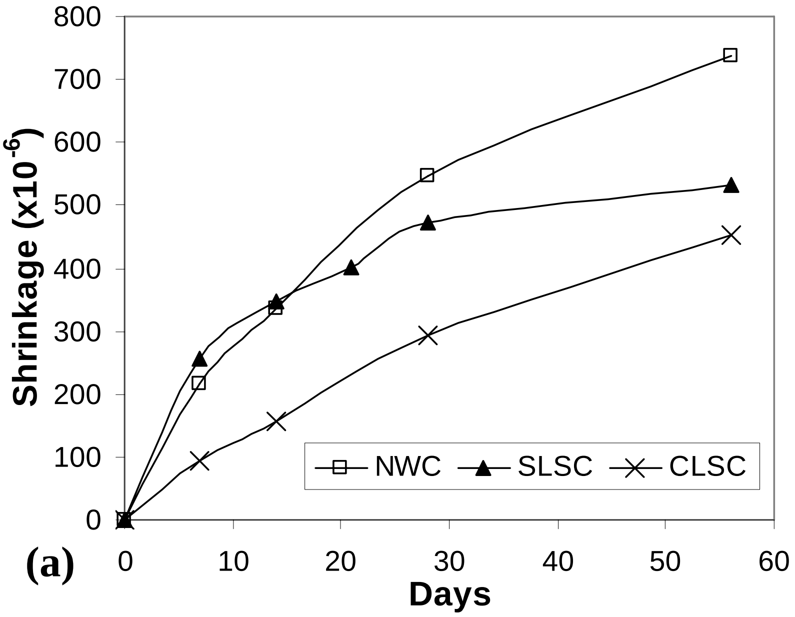
Table A1-6. Phase 5 (long-term beam tests) concrete tested properties

Beam	Concrete	Compressive strength at 28 day (MPa)
H	NWC G35	39.1
I and J	CLSC G35	38.1

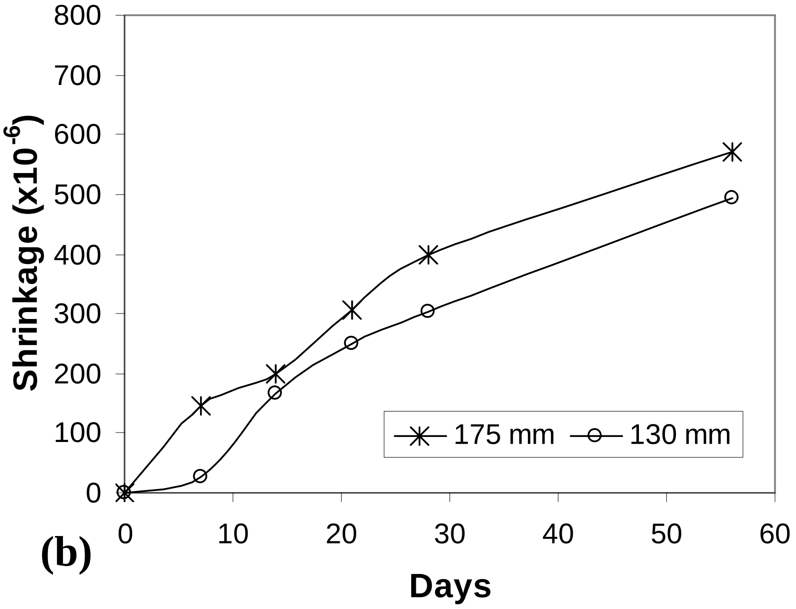
Table A1-4 gives a summary of properties tested for concrete used in Phases 1, 2 and 3 short- and long-term push-out tests. Table A1-5 presents a summary of properties tested for concrete used in Phase 4 short-term beam tests and Table A1-6 presents a summary for Phase 5 long-term beam tests. The elastic modulus of the concrete was calculated using Eq. A1-1 from Clause 5.2.3 from NZS 3101: Part 1 (SNZ, 2006) where the varying densities of the different concrete mixes were taken into account.

$$E_c = (3320\sqrt{f'_c} + 6900)\left(\frac{\rho}{2300}\right)^{1.5} \qquad \text{Eq. A1-1}$$

The average density of CLSC was 2405 kg/m^3 and the calculated elastic modulus was 33.40 GPa.



(a)



(b)

Fig. A1-1. Average shrinkage (in microstrain) vs. days for (a) Different concrete at 100 mm slump, and (b) Similar concrete but different slump

The drying shrinkage of different concrete (NWC, CLSC and SLSC) at 100 mm slump is summarized in Fig. A1-1(a). At 28 day, the drying shrinkage (in microstrain) of NWC, SLSC and CLSC were 546, 476, and 294, respectively. NWC shrank 1.8 times larger than CLSC and 1.15 times larger than SLSC. Fig. A1-1(b) shows the influence of slump on concrete drying shrinkage. At 28 day, concrete with 175 mm slump measured 400 microstrain drying shrinkage and 130 mm slump measured 304 microstrain. High slump concrete resulted higher drying shrinkage. Hence, it is crucial to ensure the desired slump is attained in order to achieve the specified drying shrinkage.

Table A1-7. Truforum LVL general specifications and characteristic properties

Species	Radiata Pine		
Grade	D		
Thickness veneer	3 mm		
Joints	3 outer plies are scarf jointed, inner plies are butt/scarf jointed		
Adhesive	Phenolic		
Density	580 kg/m^3 approximately		
Section	400d × 63w mm		
Elastic Moduli (MPa)			
Modulus of elasticity		E	10,700
Modulus of rigidity		G	660
Characteristic Strengths (MPa)			
Bending		f_b	48
Tension parallel to grain		f_t	33
Compression parallel to grain		f_c	45
Shear in beams		f_s	5.3
Compression perpendicular to grain		f_p	12
Shear at joint details		f_{si}	5.3

1.2 Laminated veneer lumber (LVL)

Laminated veneer lumber (LVL) made from Truform recipe was supplied by Carter Holt Harvey Ltd (CHH) in 400d × 63w mm sections where d and w are the depth and width, respectively. LVL is an engineered material made by laminating 3 to 3.5 mm thick veneers together in the parallel orientation to minimise the influence of defects such as knots on the strength of the member. The process eliminates any concentration or local effect of defects and thus result in a reliable and structurally stable material compared to solid or glued laminated timber. The general specifications and characteristic properties of Truform LVL are given in Table A1-7 (Gaunt and Penellum, 2004).

1.3 Derivation of LVL mean strength

The LVL joist is subjected to a combination of bending and tension. Timber has different strength and distribution in bending and tension.

The distribution of bending strength, f_b, and MOE (Apparent Modulus of Elasticity, which includes also allowance for shear deformation) were obtained from Carter Holt Harvey (CHH) factory production test in Auckland, where all the tests were on 95d × 63w mm LVL cross-sections taken from entire billets. Since the actual size of LVL used in the composite beam specimens was 400d × 63w mm, the factory data were corrected for size effect. This was done using a strength reduction factor derived by CHH for use in the USA, given by:

$$f_b(D) = \left(\frac{12}{D}\right)^{0.1538} f_b(12) \qquad \text{Eq. A1-2}$$

where D, is beam depth measured in inches. Using the above equation, a value of a strength reduction function of 0.80 was obtained from the 95d × 63w mm specimens to the 400d × 63w mm specimens. Therefore, the factory data, mean f_b = 58.43 MPa was modified for this reduction to f_b = 46.84 MPa.

The distribution of the tensile strength was obtained from CHH factory production test in USA. In this case, the strength value depends upon the length of the specimen tested, and reduces as the length increases. A relationship has been proposed to account for this effect: the tensile strength is multiplied by a strength reduction factor for length, given by:

$$f_t(L) = \left(\frac{8}{L}\right)^{0.111} f_t(8) \qquad \text{Eq. A1-3}$$

where L, is beam length measured in inches. Since no specimen among those tested was 8 m long, using the equation above, a relationship was derived between the values for specimen 3048 mm long and the f_t(8 m and 10 m). The values f_t(8 m) = 33.38 MPa and f_t(10 m) = 32.71 MPa was calculated.

For timber and LVL members subjected to a combination of bending and tension, the Eurocode 5 assumes that failure occurs when the flexural component of the stress, σ_b and the tensile component of the stress, σ_t satisfied the following equation:

$$\left(\frac{\sigma_b}{f_b}\right)+\left(\frac{\sigma_t}{f_t}\right)\leq 1 \qquad \text{Eq. A1-4}$$

The inequalities were manipulated so as to express it in terms of the maximum stress in the bottom fibre of the LVL beam, σ_{max}:

$$\sigma_{\max}=\sigma_b+\sigma_t\leq f(M/N)=\frac{1+\dfrac{\sigma_t}{\sigma_b}}{\dfrac{f_t}{f_b}+\dfrac{\sigma_t}{\sigma_b}}f_t \qquad \text{Eq. A1-5}$$

Table A1-8. Stress and strength ratios of TCC beams in Phase 4 short-term beam tests

Beam	σ_t/σ_b	*f(M/N)* in MPa
A	0.773	39.8
B	0.828	39.6
C	0.871	39.4
D	0.888	39.4
E	0.906	38.9
F	0.885	39.4
G	0.823	39.6

The tensile and bending stress ratio, σ_t/σ_b, depends on the bending moment and axial force strength ratio, *f(M/N)*, in the LVL joist, which is affected by the stiffness ratios between concrete and timber, and by the stiffness of the connection system. The gamma-method was used to investigate the stress distribution in the TCC beam, a first approximation of the stress ratio was calculated for each beam. The σ_t/σ_b values are reported in Table A1-8 together with the resulting strength f(M/N), which represents the value of the tensile stress in the bottom fibre of the LVL joist leading to failure for fracture in tension.

It was noted that the tensile stress do not significantly change for the different specimens. Therefore, an average value of *f(M/N)* = 39.4 MPa was assumed for all beam specimens. Hence, the mean combined bending and tension strength, f_{mean} = 39.4 MPa was assumed for the LVL tested as part of the composite beam.

The LVL design strength for bending, $f_{b,d}$ = 43.2 MPa and for tensile, $f_{t,d}$ = 27 MPa was calculated by multiplying the characteristic properties in Table A1-7 by the strength reduction factor, ϕ = 0.9:

$$f_{b,d} = \phi f_{b,k} \qquad \text{Eq. A1-6}$$

$$f_{t,d} = \phi f_{t,k} \qquad \text{Eq. A1-7}$$

The maximum design strength in the LVL subjected to tension and bending, f_d = 33.85 MPa was calculated using the following formula where σ_t/σ_b values taken from **Table A1-8**:

$$f_d = \frac{1 + \dfrac{\sigma_t}{\sigma_b}}{\dfrac{f_{t,d}}{f_{b,d}} + \dfrac{\sigma_t}{\sigma_b}} f_{t,d} \qquad \text{Eq. A1-8}$$

1.4 Interlayer plywood and reinforcement

17 mm thick plywood (Grade F8 Ecoply) was used as a permanent formwork for the specimens. The choice of the plywood thickness corresponds with the formwork strength requirement of timber-concrete composite beams which were built for other phases of the research. High tensile 10 mm diameter deformed steel reinforcement spaced at 200 mm centres on both ways was used mainly for cracking control.

APPENDIX

2. Concrete material testing

This appendix describes tests carried out particularly on concrete material which were used to construct the push-out and beam specimens. These tests were: (1) Compressive strength using cylinders; (2) Workability using slump test; and (3) Drying shrinkage using prisms.

2.1 Cylinder compression test

The number of 100 × 200 mm cylinders made was as such that there were a minimum of 3 cylinders per compression test at 7 days, 28 days and at the day the push-out specimen was tested. Cylinder compression tests were performed according to New Zealand Standard NZS 3112: Part 2 (SNZ, 1986a). The aim of this test is to determine the strength of concrete. The apparatus required and procedures are as follows:

1. Cylinder moulds made of metal castings capable of being opened longitudinally to facilitate removal of the specimen without damaging them and fitted with efficient closing devices that will ensure accurate alignment and tight closure of the joints in service. The moulds come with machined metal base and top plates that can be screwed tightly. A few hours before the moulds are used, they were dipped into a tank of lubricant oil and subsequently oil was allowed to drain off leaving a thin layer of lubricant to ensure easy removal of the cylinders (Fig. A2-1(a));
2. A tamping rod measuring approximately 16 mm in diameter and 600 mm in length, with one end rounded to a hemispherical tip. The cylinders were filled up in 3 layers with each layer compacted 25 times to its depth using the tamping rod. The concrete was even out in the final layer and tightly closed with the top plate. Concrete stains on the external cylinder were washed off immediately and the moulds were laid on their sides in a horizontal position for 24 hours with the longitudinal lid facing upward;
3. Alternatively, a vibrating table can be used for the purpose of compaction. In this method the concrete was poured only in 2 layers and each layer vibrated until bubbles appeared on the surface of the concrete. This normally takes just a few seconds (Fig. A2-1(b));

4. The cylinders moulds were removed after 24 hours (Fig. A2-1(c)) and the cylinders were placed in a fog room where they were kept moist for curing.

At the day of testing of the cylinders, they were taken out of the fog room. The top surface of the cylinder and sides were polished using a pumice stone in order to obtain an even surface checked with a square rule (Fig. A2-1(d)). The cylinder was positioned centrally on the lower platen of a Universal Testing Machine with a steel cap on top of the cylinder to avoid concentration of load (Fig. A2-1(e) to (g)). Each cylinder was tested to failure at a 23 kN/min constant loading rate and the maximum failure load recorded. The compressive strength of the cylinders in MPa was calculated by multiplying the maximum load expressed in kN with a factor of 0.1275.

2.2 Slump test

A slump test was first carried out to check the workability of the concrete before starting the concrete pour. The slump test provides a rapid method for determining the relative consistencies of successive batches of mixed concrete. The required specified slump was 120 mm being a slump measure commonly used for the construction of floors in New Zealand which is the final purpose of the tested connection specimens. Concrete failing to meet the acceptable range were declined. The apparatus required for the slump test comprises of: (1) a slump cone made of metal, with an opened base and top fitted with lifting handles and foot pieces on the sides; (2) a circular tamping rod made of steel, approximately 16 mm in diameter and 600 mm long; and (3) scoop, steel rule, and trowel.

Fig. A2-1. (a) Draining off oil from cylinder moulds; (b) Casting of cylinders and vibration table; (c) Cylinders and prisms kept in fog room; (d) Universal testing machine for cylinder test; (e) Preparation of cylinder before test; (f) Positioning cylinder at centre of lower platen; and (g) Cylinder after test.

The slump test was performed in accordance to the procedure described in the New Zealand Standard NZS 3112: Part 1 (SNZ, 1986b). The inner surface of the slump cone was first wet with a piece of damped cloth and placed on a flat, moist, non-absorbent, rigid surface which was level and free from vibration (Fig. A2-2(a)). The concrete was carefully poured in 3 separate layers into the slump cone which was hold down firmly with the feet (Fig. A2-2(b)). Each layer of concrete was compacted uniformly covering the whole surface and only on each respective layer with 25 blows using the tamping rod. After compacting the third layer, the cone is topped up with additional concrete and then flattened off gently with a trowel. Holding the cone down firmly, the surface around the cone was cleaned. The cone was steadily and vertically lifted upward in 3 counts with no lateral or torsional motion. Immediately after the removal of the cone, the slump was measured by determining the difference between the height of the cone and that of the highest point of the slumped concrete (Fig. A2-2(c)).

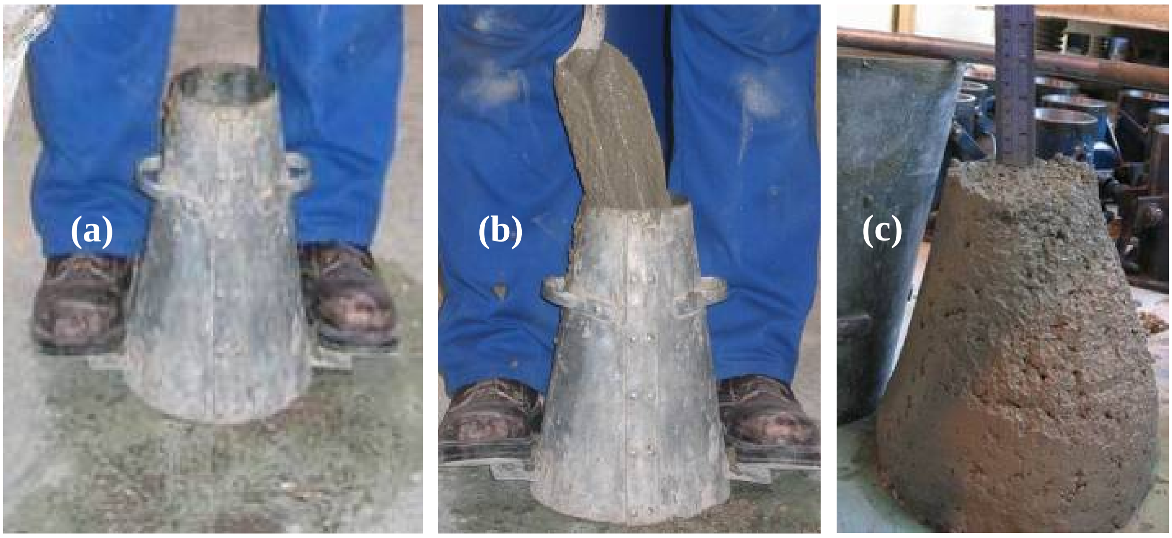


Fig. A2-2. (a) Moist surface preparation; (b) Cone held firmly by legs as concrete was inserted into the cone; and (c) Measure of slump.

2.3 Prisms for drying shrinkage test

Concrete dry shrinkage test was performed on concrete prisms moulded and kept under specific conditions as described the New Zealand Standards NZS3112: Part 3 (SNZ, 1986c). It provides a method for determining the length changes of the specimens due to drying air. The apparatus used were:

1. Steel moulds to make prisms measuring 75*b* × 75*d* × 300*l* mm where *b*, *d*, and *l* are breath, depth, and length, respectively, with their internal surfaces smoothly

finished and substantially leak-proof. Each mould shall be provided with a base plate to which two end plates are securely fastened by screws; two side plates which are fastened to the ends plates by screws, and two partially loose end plates which act as gauge stud holders. The gauge studs on the ends of the mould shall be secured tightly and positioned to provide a gauge length of 250 mm measuring between the internal tips of the studs. Each gauge stud holder shall be held in position against the end plate by a retaining screw and shall be capable of release after compaction of the concrete;

2. Vibrating table used to compact the concrete in the moulds;
3. Conditioning chamber to store the specimens at controlled temperature of 23±2°C and a relative humidity of 50±5°C at all times;
4. A length comparator capable of measuring the change of length in the specimens with a precision of 0.001 mm.

Fig. A2-3. (a) Prisms in the mould; (b) Comparator to measure the prisms change in length.

For each concrete casting, at least 2 prisms were poured together with the cylinders for the purpose of this test (Fig. A2-3(a)). The prisms were removed from their mould after 24 hours leaving the gauge studs which were cast into the ends of each prism. Immediately after, the prisms were stored in the fog room to cure for the period of 7 days before they were taken out, surface wiped dry and quickly stored in a conditioning chamber. Prior to placement in the conditioning chamber, each prism was positioned in the comparator (Fig. A2-3(b)) so that its axis was aligned with the measuring anvil so as to measure the initial length. In order to obtain a correct measure of length, the prism was adjusted 2-3 times with a slight horizontal push until the reading maintains and hence recorded to 0.001 mm of accuracy. This process was repeated each time a reading was taken.

The prisms in the conditioning chamber were positioned so that there was a clearance of at least 50 mm on all sides allowing clear aeration of the prisms. Length measurements were taken for each prism at 0, 7, 14, 21, 28, 56, 90 and 120 days. The temperature and the relative humidity of the chamber were also recorded. On the basis of these data the drying shrinkage could be calculated:

1. For each reading taken, the change in length was tabulated as the measured length measurement subtracted from the initial length measurement at day 0;
2. The change in length tabulated in (1) was divided by the effective gauge length, taken as 250 mm;
3. The drying shrinkage hence is, expressed in microstrain ($\times 10^{-6}$).

APPENDIX

3. Construction of push-out specimens

This appendix describes in detail the method used to construct all the push-out specimens involved in the project.

3.1 General

Short-term symmetrical push-out tests were carried out in two phases from the end of 2006 to 2008 at the University of Canterbury, New Zealand. A long-term push-out test was performed in conjunction with the second phase of the short-term push-out test. A total of 30 specimens of 15 connection types (A1 to C2 detailed in Fig. A3-1(a) to (f), and, D1 to H4 in Fig. A3-2(a) to (e)), 2 specimens of each were constructed in the first phase of the push-out test while 36 specimens were built in the second phase of the push-out test which was made up of 30 specimens for the short-term test and 6 specimens for the long-term test. 3 types of connections were constructed in the second phase, 9 specimens of each as detailed in Fig. 4-1 (in the main text): (1) triangular notched coach screw – T; (2) 300 mm rectangular notched coach screw – R; and (3) toothed metal plate – P. Another 3 specimens of the triangular notched connection were built to be tested in the weak direction (TT). It took approximately 2 months to make all the specimens in each phase. All the construction activities were carried out in the Structures Laboratory except for the pressing of the plates into the LVL for specimens with toothed metal plate connection which was performed at a Mitek fabricator – Westlake Timber located in Christchurch.

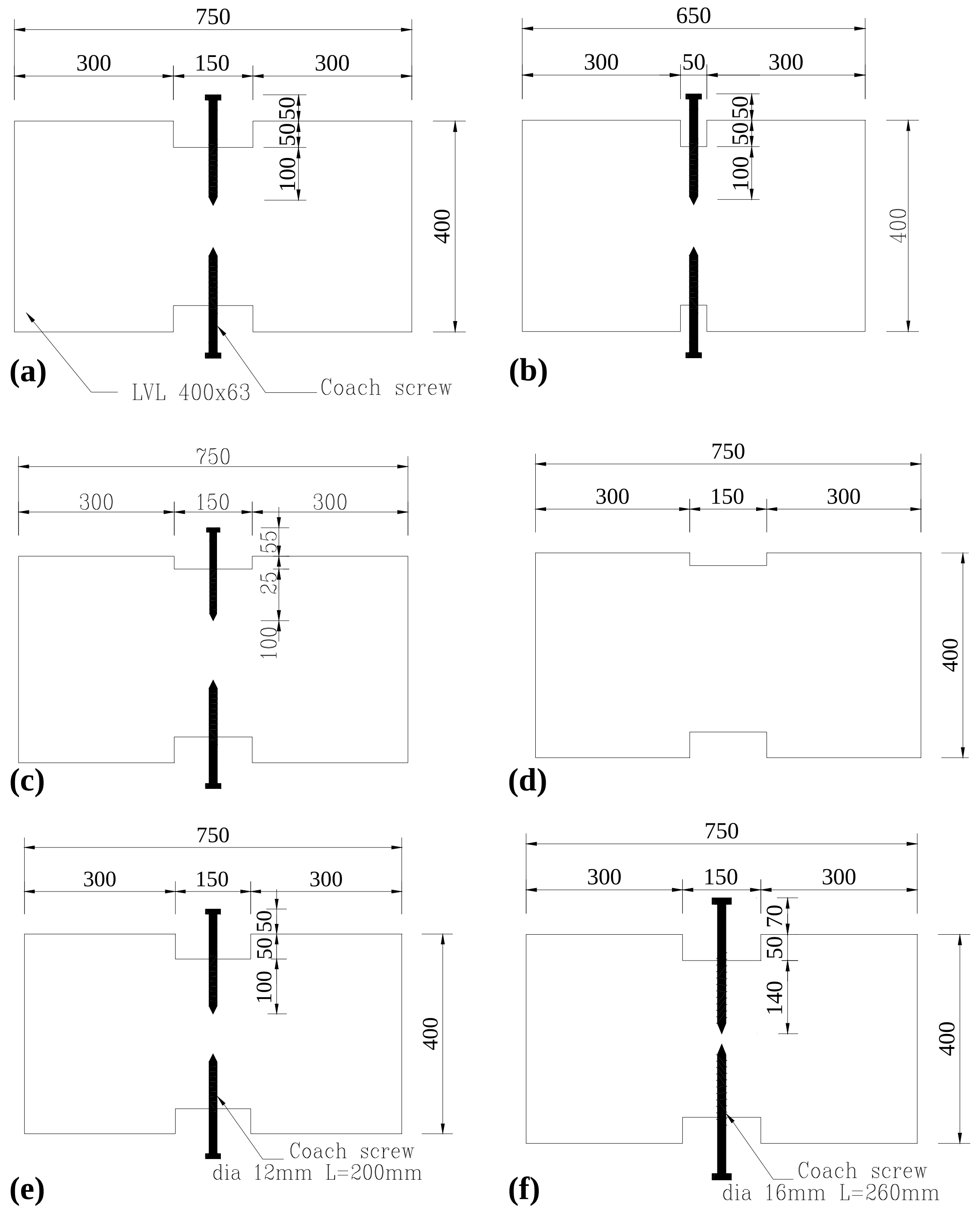


Fig. A3-1. First phase push-out test specimens, A1 to C2: (a) A1- Rectangular notch 150×50×63 mm coach screw ϕ16; or G1 - similar to A1 but with low shrinkage concrete; or H1 - similar to A1 but a double LVL; (b) A2 - Rectangular notch 50×50×63 mm coach screw ϕ16; (c) A3 - Rectangular notch 150×25×63 mm coach screw ϕ16; (d) B1-Rectangular notch 150×50×63 mm; (e) C1: Rectangular notch 150×50×63 mm coach screw ϕ12; and (f) C2: Rectangular notch 150×50×63 mm coach screw ϕ16 depth 140 mm.

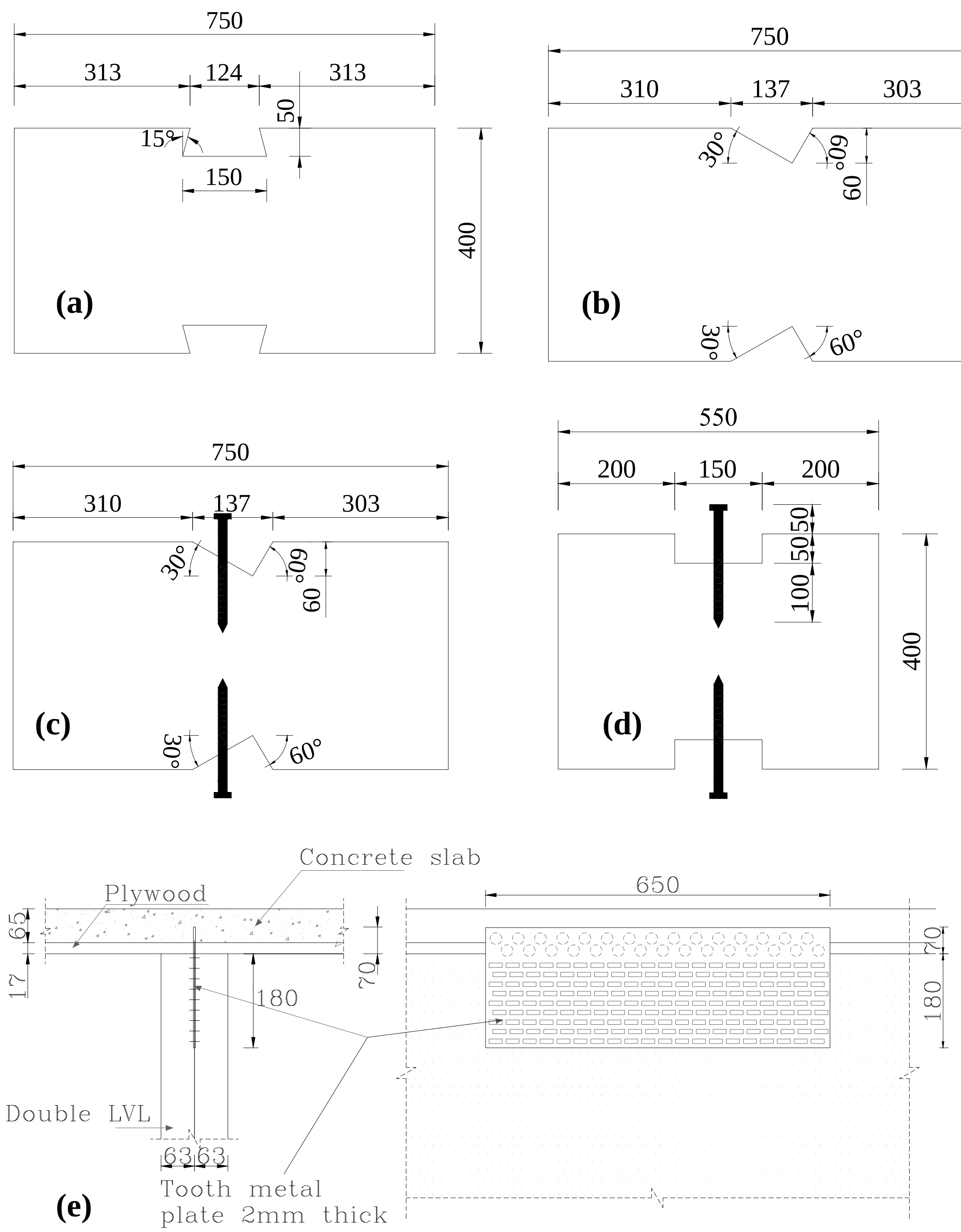
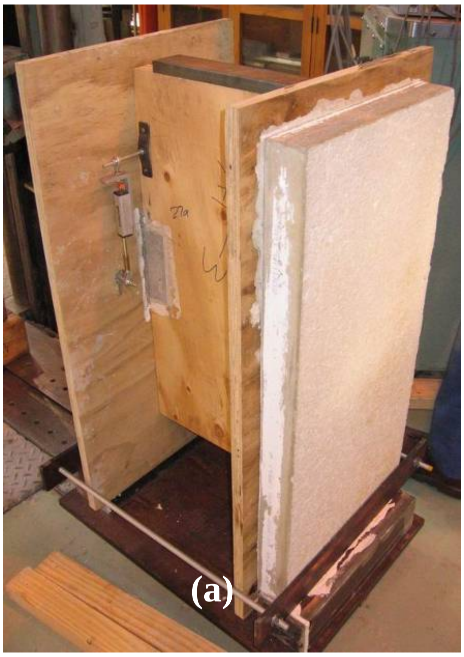


Fig. A3-2. First phase push-out test specimens, D1 to H4: (a) D1 - Doves tail notch 150×50×63 mm; (b) E1 - Triangular notch 30°_60° 137×60×63 mm; (c) E2: Triangular notch 30°_60° 137×60×63 mm coach screw ϕ16; (d) F1: Rectangular notch short end 150×50×63 mm coach screw ϕ16; (e) H2, H3, and H4 – Toothed metal plate of length 650, 325 and 150 mm, respectively.

The main components of a test specimen were the concrete slab and the LVL beam which was connected either by a notch with or without coach screw, or a toothed metal plate. The construction of the test specimens involved the following steps: (1) formwork making for the slab; (2) cutting of the LVL and the notches; (3) insertion of the coach screws; (4) assembly of slab formwork to LVL beam; (5) reinforcing works; and (6) casting of concrete. Approximately two months were taken to prepare all test specimens in each push-out test phase by two skill workers working 8 hours a day, 5 days a week before they were ready for concreting. Hence, 640 man-hours were utilised to construct 30 numbers of symmetrical push-out test specimens in a laboratory setting where most of the works were done manually or with small power tools. Fig. A3-3 illustrates a typical push-out test specimen.



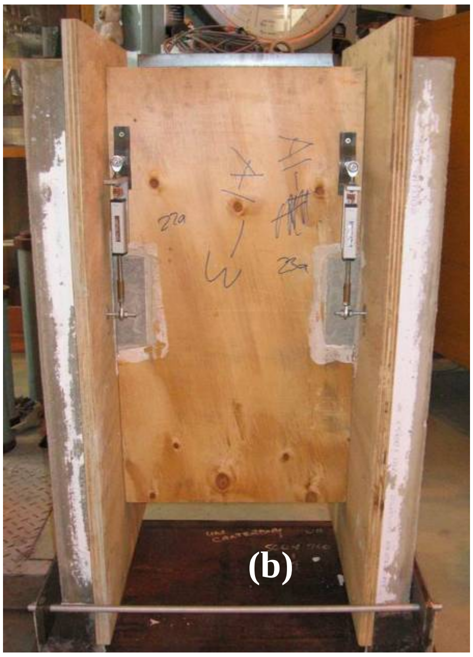


Fig. A3-3. Typical push-out test specimen: (a) 3 dimensional view; and (b) Front elevation

3.2 Formwork making for the slab

45*b* × 90*d* mm sawn timbers were used to make the edges of the slab formwork while 17 mm thick plywood for the formwork base. The two components were interconnected by underlying splices connected with screws (Fig. A3-4(a)). A total of 200 linear meter 45*b* × 90*d* mm sawn timbers and 25 m^2 of plywood were used in the second phase push-out test. All the sawn timber and plywood were first measured and cut to the required length and size before they were fabricated into a box (Fig. A3-4(b)).

Fig. A3-4. (a) Preparation of plywood base interconnected with screws to the edge forms; (b) Box formwork for the concrete slab; (c) Cutting of notches using bandsaw; (d) 200 mm length coach screw.

Rectangular holes for the notches were cut in the plywood using a jigsaw. Two 28 mm diameter holes were also made adjacent to the middle length of the connection on both sides for instrumentation purposes. 6 mm threaded rod with coupling nuts was inserted through each hole extending to the mid depth of the concrete slab and cast into the concrete to enable the mounting of potentiometers during the testing. Once the boxes were ready, they were painted with two layers of basic white acrylic paint in order to prevent absorption of water from the concrete into the timber. This also avoid the lost of water from the concrete which can cause excessive drying shrinkage and cracking.

3.3 Cutting of LVL and notches

Table A3-1 shows the dimensions and numbers of LVL used to build test specimens for the second phase of the push-out test. In both phases of the push-out test, the LVL were first measured, marked, and cut to length before the notches were removed from both longitudinal sides using a band saw (Fig. A3-4(c)). The operation was tedious and time consuming as it meant cutting along a 63 mm thickness of the LVL. For all the connection types, single LVL were required except for the toothed metal plate connection where double LVL were used so that the plates can be sandwiched in between the two pieces of LVL.

Table A3-1. Dimensions and numbers of LVL pieces used to construct the specimens in second phase of push-out test

Type of connection	Numbers	Dimensions $b \times d \times l$ (mm)
Triangular notch (T and TT)	14	63 × 400 × 660
Rectangular notch (R)	11	63 × 400 × 805
Toothed metal plates (P)	22	63 × 400 × 760

3.4 Insertion of coach screws

Prior to the insertion of the coach screw using a socket wrench, a hole to the size of the shank of the coach screw (12.5 mm in diameter) had to be predrilled. The coach screws which were obtained from Blacks Fasteners were of varying sizes and lengths for the first phase while in the second phase only 16 mm diameter and 200 mm length coach screws were used (Fig. A3-4(d)). In the second phase, all the coach screws were embedded 100 mm into the LVL from the surface of the notch at the middle of the notch length and breadth (Fig. A3-5(a)).

3.5 Pressing of toothed metal plates

The toothed metal plate used in the second phase were specially designed, pressed and supplied by Mitek for this timber-concrete composite project. The 1 mm thick plate is 136d × 333l mm with 2 rows of 22 mm diameter holes on the upper section and 1.2 mm length teeth on the remaining lower section of 86 mm depth (Fig. A3-5(b)). The holes enabled concrete bonding with the plate and for the reinforcing steel to protrude across the slab. The pressing of the plate into the LVL were carried out in Westlake Timber, a licensed fabricator of Mitek. Each LVL were marked before placing and pressing the plates into the LVL (Fig. A3-5(c) and (d)).

Fig. A3-5. (a) Coach screw inserted into triangular notch; (b) Specially designed tooth metal plate; (c) and (d) Pressing of plates into LVL

3.6 Assembly of slab formwork to LVL

Two to three $90l \times 3.15\phi$ mm head-collated nails using a nail gun temporarily fasten each plywood box to the LVL on both sides. The nails should not contribute to the strength of the connection. The notch openings at the underside of the plywood panels were covered with small plywood remnants. In order to ensure stability of the formwork slab flanges, a piece of sawn timber was screwed to the top and bottom flanges (Fig. A3-6(a)).

For the specimens with toothed metal plate, 2 LVL pieces were first nailed together, 3 nails on each side of the LVL face in a staggered orientation as oppose to the other face of the LVL (Fig. A3-6(b)). Next, the plywood boxes were fastened onto the double LVL on both longitudinal sides with a minimum number of nails. A 1 to 3 mm gap between the 2 pieces of LVL that were nailed together is inevitable due to the thickness of the plates and slight eccentricity that may have been present caused by the pressing in the plates (Fig. A3-6(c)).

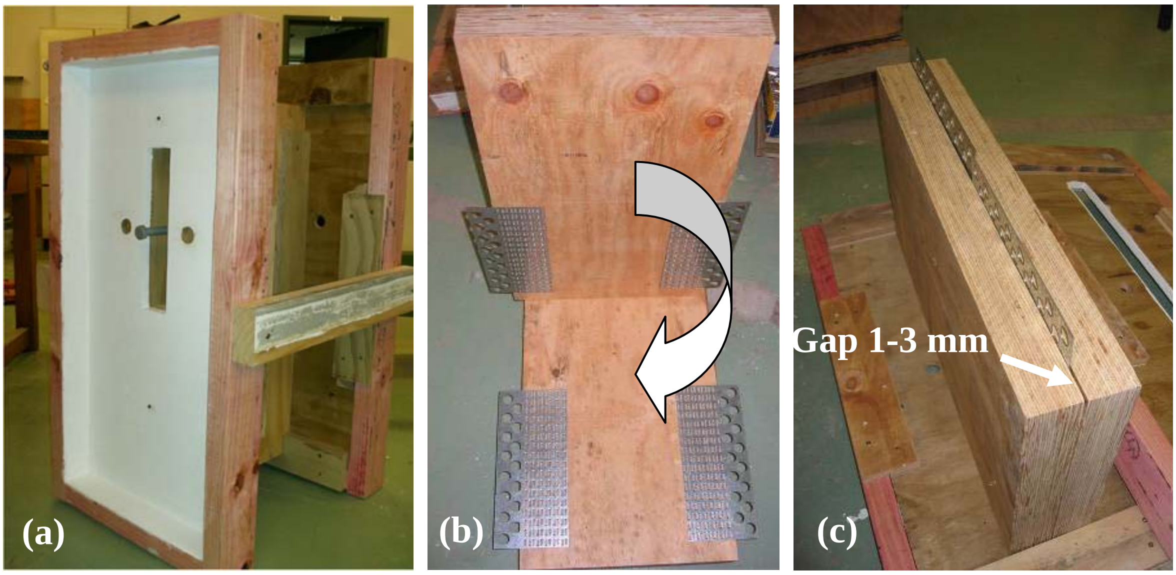


Fig. A3-6. (a) Rectangular notched connection specimen completely assembled, also showing surface painted with white acrylic paint; (b) Staggered orientation of plates in LVL; and (c) Toothed metal plate connection specimen with double LVL

3.7 Concreting of the specimens

Fig. A3-7(a) and (b) show the specimens in the second phase ready for concrete pouring. The concrete was poured in two sessions for each batch as the specimens had to be turned over after the casting of the first side of the slab for the other side to be poured after seven days (Fig. A3-7(c) to (f)). The concrete was compacted using a poker vibrator (Fig. A3-7(d)). Care was taken so that the concrete in the notches are well compacted. Each side of the slab was cured once after the concrete hardened (after six to seven hours) by means of damp Hessian sacks for four to five days (Fig. A3-7(e)). The curing process was important in order to reduce the rate of hydration and thus prevent the concrete from cracking.

Fig. A3-7. (a) Toothed metal plate connection specimen; (b) Specimens awaiting concrete; (c) Concrete pouring of specimens; (d) Compacting of concrete with a poker vibrator; (e) Concrete curing with damp Hessian sacks; (f) Specimens after curing stage (top view)

APPENDIX

4. Notched connection strength evaluation analytical model

This appendix presents the calculation spreadsheet for the strength evaluation analytical model of notched coach screw connection which has been discussed in Section 4.7 of the main text.

4.1 Analytical model to New Zealand Standard

The analytical model was derived based on four possible failure modes:

1. Failure of concrete notch in shear (Fig. A4-1)
2. Failure of concrete notch in crushing (Fig. A4-2)
3. Failure of LVL in longitudinal shear (Fig. A4-3)
4. Failure of LVL in crushing parallel to the grain (Fig. A4-4)

Based on the push-out tests, in all case, it was the failure of concrete notch in shear that governed.

FAILURE OF CONCRETE IN SHEAR

Notch shear strength	$0.2f'_c bl =$	170.1	kN
NZS 3101 Design of corbels (p.16-2)			
Coach screw strength	$nk_1 pQ_m =$	16.33	kN
NZS 3603 (p.62)			
TOTAL STRENGTH ($F_{conc\ shear}$)	$0.2f'_c bl + nk_1 pQ_m =$	$\boxed{186.4}$	kN
Notch length	$l =$	300	mm
Notch breadth	$b =$	63	mm
Concrete mean compressive strength	$f'_c =$	45	Mpa
Number of coach screw	$n =$	1	
Modification factor for load duration	$k_1 =$	1	
NZS 3603 (p.22) (not green timber; short duration loading; not in end grain)			
Penetration or anchor length	$p =$	100	mm
Screw characteristic withdrawal strength	$Q_k =$	147	N/mm
NZS 3603 (p.62) (group **J4**); the shank diameter of the coach screw is 12.5mm			
Screw mean withdrawal strength	$Q_m =$	163.33	

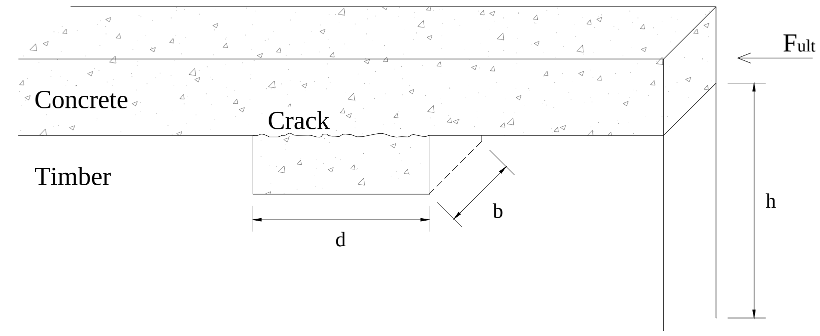


Fig. A4-1. Failure of concrete notch in shear along length

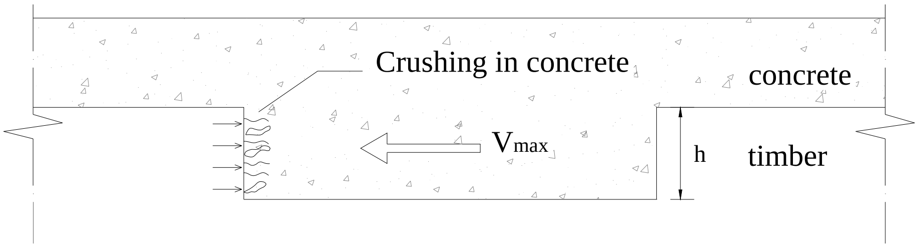


Fig. A4-2. Failure of concrete notch in crushing

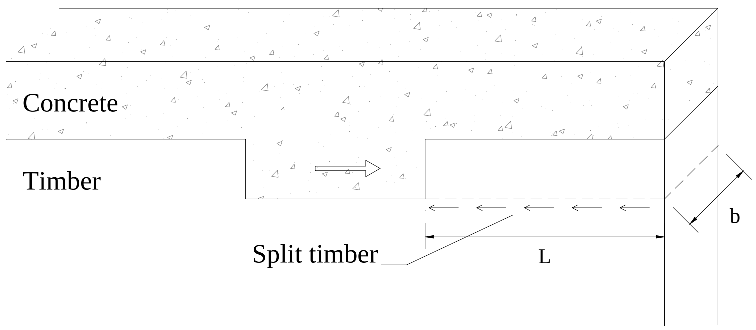


Fig. A4-3. Failure of LVL in longitudinal shear

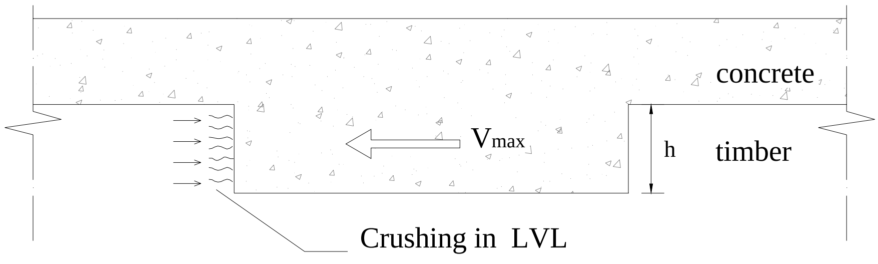


Fig. A4-4. Failure of LVL in crushing parallel to the grain

FAILURE OF CONCRETE IN CRUSHING

Concrete crushing strength	$F_{conc\ crush} = f'_c A_c$	$\boxed{141.8}$	kN
NZS 3101 (p.16-1)			
Notch depth	$h =$	50	mm
Notch breadth	$b =$	63	mm
Effective area	$A_c =$	3150	mm^2
Concrete mean compressive strength	$f'_c =$	45	Mpa

FAILURE OF LVL IN SHEAR

LVL longitudinal shear strength ($F_{LVL\ shear}$)	$k_1 k_4 k_5 f_s Lb =$	133.56	kN
LVL mean longitudinal shear strength	$F_{LVL\ shear} / \phi$	$\boxed{148.4}$	kN
NZS 3603 (p.21)			
LVL characteristic shear strength	$f_s =$	5.3	Mpa
LVL shear length	$L =$	400	mm
Notch breadth	$b =$	63	mm
Modification factors			
Load duration (for short-term)	$k_1 =$	1	
Load sharing (for LVL)	$k_4 =$	1	
	$k_5 =$	1	
LVL Strength reduction factor	$\phi =$	0.9	

FAILURE OF LVL IN CRUSHING

LVL compressive strength ($F_{LVL\ crush}$)	$k_1 f_c bd =$	141.75	kN
LVL mean compressive strength	$F_{LVL\ crush} / \phi$	$\boxed{157.5}$	kN
LVL characteristic compressive strength	$f_c =$	45	Mpa
Load duration (for short-term)	$k_1 =$	1	
Notch depth	$d =$	50	mm
Notch breadth	$b =$	63	mm

4.2 Analytical model to Eurocode

FAILURE OF CONCRETE IN SHEAR

Concrete **EC2 (p.86)**

b_n =	63	mm
l_n =	300	mm
f_{ck} =	35	MPa
$v = 0.6(1 - f_{ck}/250)$	0.516	MPa
$f_{cd} = f_{ck}/1.6$	21.88	MPa
f_{cm} =	45	MPa
$v^* = 0.6(1 - f_{cm}/250)$	0.49	MPa
$a_v = 0.5d$ =	150	
$\beta = a_v/2d$	0.25	

when the distance of the loading point is less than a_v

$\beta^* = (l_n - 2\phi_{cs})/2l_n$	0.45	

coefficient considering the notch length and outer diameter of coach screw

LVL + Coach screw **EC5 (p.77)**

n_{ef} =	1	coach screw
ϕ_{cs} =	16	mm outer diameter
d_p =	100	mm
$d_{ef} = d_p - \phi_{cs}$	84	mm
ρ_k =	580	kg/m^3
$\rho_m = \rho_k / 0.9$	644.44	kg/m^3
$f_{w,k}$ =	50.29	MPa

characteristic withdrawal strength perpendicular to grain

$f_{w,m} = f_{w,k}/0.9$	58.90	MPa

EC method

$\beta 0.5 b_n l_n v^* f_{cm}$	52.31	kN
$n_{ef}(\pi \phi_{cs} d_{ef})^{0.8} f_{w,m}$	46.83	kN
TOTAL	99.14	kN

EC* method

$\beta^* 0.5 b_n l_n v^* f_{cm}$	93.45	kN
$n_{ef}(\pi \phi_{cs} d_{ef})^{0.8} f_{w,m}$	46.83	kN
TOTAL	140.28	kN

APPENDIX

5. Short-term connection push-out test

This appendix presents the setup, computation of results, and photographs (for Phases 1 and 2) of short-term connection push-out tests.

5.1 Connection push-out test setup

The connection push-out test setup which was performed in the Structures Laboratory of the Department of Civil and Natural Resources Engineering, University of Canterbury is illustrated in Fig. 3-6 (in main text) and described in Section 4.4 (in main text). 50 mm potentiometers with ±0.4% accuracy: P1, P2, P5 and P6 (Fig. A5-1(a and b)) were used to measure relative slips in connection potentiometer P3 for horizontal slip (Fig. 3-6 in main text). Horizontal slip was measured to observe possible separation between LVL and concrete and to aid in determining whether there would be a problem with the flanges of the specimens separating from the LVL web and falling out, possibly damaging equipment or injuring someone. One specimen was tested without any form of restraint, and it was found that significant separation occurred due to the bending moment induced in the specimen by the load. Since in the actual timber-concrete composite beam the connections are loaded in pure shear and separation between the concrete and timber is not likely to occur, it was decided to prevent the separation in the specimens using steel straps. These were placed at the top and bottom of the specimens for two tests and then the bottom strap only was used for another two tests. It was found that the strength values were almost identical for having two restraints or only one restraint at the bottom; therefore it was decided to just use the single strap at the bottom of the specimens for the rest of the testing.

A potentiometer was used at location P4 (Fig. 3-6 in main text) to compare the difference between measuring the slip at the connection or at the base. This was because previous push-out tests performed by Kuhlmann and Michelfelder (2004) used this form of measurement, and comparison between the results obtained at the connection level and at the base plate will give an indication of the accuracy of this method. No significant difference in the measured slips was found. Hence, potentiometer P4 was not used in subsequent tests.

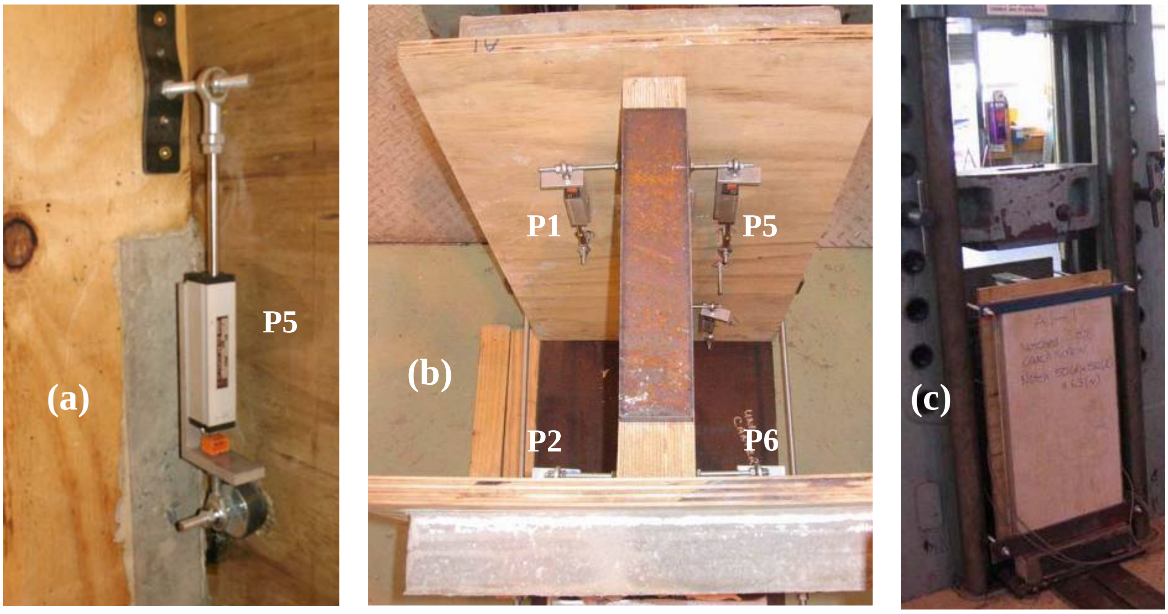


Fig. A5-1. Setting up of push-out test under a Universal Testing Machine: (a) 50 mm potentiometer used to measure relative slip; (b) 20 mm thick steel plate on top of LVL for distribution of load; (c) Specimen slid under loading ram ready for test.

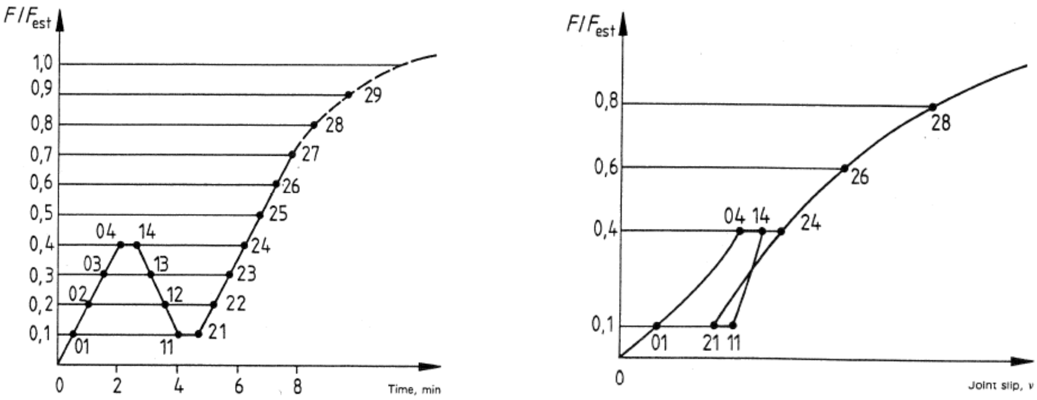


Fig. A5-2. (a) Loading procedure and (b) idealised load-slip curve

All the instrumentation such as the load cells, data acquisition boxes and potentiometers were first calibrated to ensure accuracy in the data recorded. For instance, a scale factor was found for each of the calibrated potentiometers. This scale factor was multiplied by a coefficient measured in the calibration to return the correct measured displacement. A text file containing all the specified channels and corresponding scale factors was written and uploaded into a software known as Universal Data Logger (UDL). This UDL software was used to record the load-relative slip relationships for every channel of each test. Similarly, the load cell which is inherent in the Universal Testing Machine was calibrated as such to synchronize the digital read out in the UDL software with the original analog read out of the Universal Testing Machine.

Each specimen which weighed between 100 to 150 kg was positioned on a specially made test platform with the assistance of an overhead crane. The test platform made from steel was placed on the rail of the Universal Testing Machine outside the loading ram. The specimen was seated as such that only the concrete flanges of the specimen were supported and not together with the plywood section. A 20 mm thick steel plate was placed on top of the LVL web to facilitate an even distribution of the load (Fig. A5-1(b)). The length of this plate was made to be the distance between the bottom edges of the two opposite notches cut in the LVL and the width of the plate equal to the width of the LVL piece. Once all the potentiometers have been mounted and the specimen placed correctly, the test platform with the specimen on it was slid carefully under the loading ram of the Universal Testing Machine (Fig. A5-1(c)). A stopper knob was inbuilt onto the test platform so that when the platform is slid under the ram, it would stop in the centre position automatically and any eccentricity eliminated.

The connections were loaded at a rate of $0.2F_{est}$ kN per minute in shear with the load applied onto the LVL web section of the specimen until the connection failed. The load regime followed for the testing is presented in Fig. A5-2(a) as given in EN 26891 (CEN, 1991). This involves an initial estimate of the strength (F_{est}) of the specimen which was determined on the basis of experience, preliminary tests, or calculation. This was then adjusted for the second specimen using the new actual F_{est} from the first tested specimen. The specimen was first loaded to $0.4F_{est}$ and held for 30 seconds, then unloaded to $0.1F_{est}$ and maintained for 30 seconds. Thereafter the specimen was loaded to failure or to a maximum slip of 20 mm, whichever occurred first.

The purpose of the initial load-unload phase was to eliminate any internal friction in the connection, and to ensure any initial slip or slack present in the connection does not affect the final results. The slip measurements v_{01}, v_{04}, v_{14}, v_{11}, v_{21}, v_{24}, v_{26} and v_{28} shown in Fig. A5-2(b) were recorded for each test specimen using potentiometers that were mounted adjacent to the connections (Fig. A5-1(a)). The slip at maximum load, F_{max}, defined as the shear strength was also recorded.

Table A5-1 Estimated values used to apply the loading procedure.

Types of connection (length × depth × width) with all dimensions in mm	F_{est} (kN)	$0.4F_{est}$ (kN)	$0.1F_{est}$ (kN)	Load rate (kN/min)
Connections tested in Phase 1:				
A1: Rectangular notch 150×50×63 Coach Screw ϕ16	180	72	18	0.072
A2: Rectangular notch 50×50×63 Coach Screw ϕ16	100	40	10	0.040
A3: Rectangular notch 150×25×63 Coach Screw ϕ16	130	52	13	0.052
B1: Rectangular notch 150×50×63	100	40	10	0.040
C1: Rectangular notch 150×50×63 Coach Screw ϕ12	150	60	15	0.060
C2: Rectangular notch 150×50×63 Coach Screw ϕ16 depth 140	180	72	18	0.072
D1: Doves tail notch 150×50×63	100	40	10	0.040
E1: Triangular notch 30°_60° 137×60×63	100	40	10	0.040
E2: Triangular notch 30°_60° 137×60×63 Coach Screw ϕ16	150	60	15	0.060
F1: Rectangular notch short end 150×50×63 Coach Screw ϕ16	150	60	15	0.060
G1: Rectangular notch LSC 150×50×63 Coach Screw ϕ16	140	56	14	0.056
H1: Rectangular notch double LVL 150×50×126 Coach Screw ϕ16	260	104	26	0.104
H2: Double toothed mp 650 mm	300	120	30	0.120
H3: Double toothed mp 325 mm	200	80	20	0.080
H4: Double toothed mp 150 mm	100	40	10	0.040
Connections tested in Phase 2:				
TT: Triangular notch 30°_60° 137×60×63 Coach Screw ϕ16, in the weak direction	138	55	14	0.055
T: Triangular notch 30°_60° 137×60×63 Coach Screw ϕ16, in the strong direction	165	66	16.5	0.066
R: Rectangular notch 300×50×63 Coach Screw ϕ16	240	112	28	0.112
P: Toothed metal plate 2×333 staggered	240	112	28	0.112

The load range set on the Universal Testing Machine was 500 kN and therefore the loading rate adjusted on the machine was calculated in kN/min as $\frac{0.2F_{est}}{500}$. Table A5-1 summarizes the types of connection tested with their respective initial estimate of strength for a pair of connections, F_{est}, in the loading rate computation.

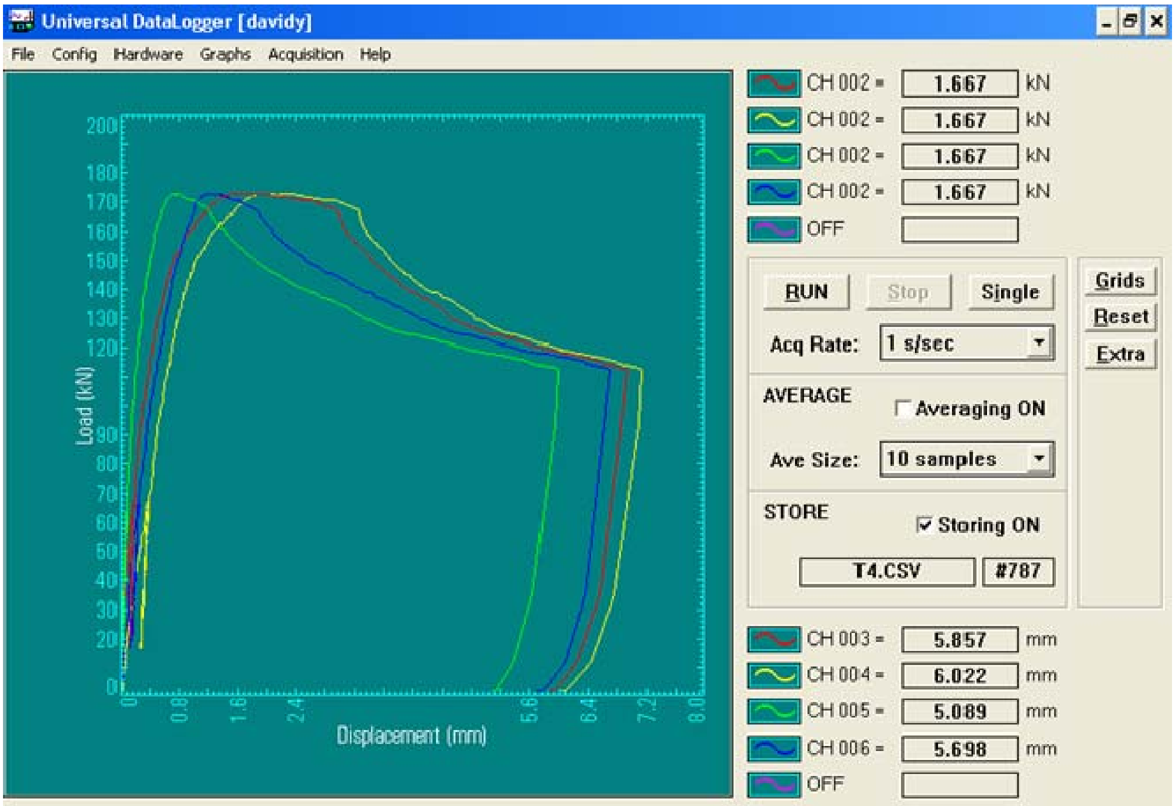


Fig. A5-3. Load-relative slip of push-out test plotted automatically in the UDL software interface

5.2 Computation of push-out test results

EN 26891 (CEN, 1991) regulates the derivation of the connection's shear strength and secant slip moduli at 40% (taken as the serviceability limit state - SLS), 60% (taken as the ultimate limit state - ULS) and 80% (at collapse) of the shear strength as outlined in Eq. A5-1 to Eq. A5-3 respectively. Eq. A5-4 to Eq. A5-9 present the calculation of different slip measurements from the push-out test that led to the computation of the aforementioned secant slip moduli. To develop a load-relative slip curve for each specimen the slip readings for the four potentiometers (P1, P2, P5 and P6) at the connection level were averaged. This load-relative slip curve was also plotted automatically in the UDL software as the specimen was loaded as presented in Fig. A5-3. Load-relative slip curves plotted in the test were for two connections. All the required parameters were computed in an Excel spreadsheet as shown in Table A5-2.

Table A5-2. Typical example of strength and slip moduli computation for a single connection in an Excel spreadsheet

		Units	A1-1	A1-2	A1-3	A1-4	AVE
F_{est}		N	65.500	65.500	90.000	65.500	
F_{max}		N	69.870	78.020	65.540	78.410	72.960
v_{01}	*of* F_{est}	mm	0.041	0.019	0.060	0.016	
v_i	$= v_{04}$ *of* F_{est}	mm	0.339	0.248	0.430	0.220	
v_{14}	*of* F_{est}	mm	0.374	0.279	0.499	0.239	
v_{11}	*of* F_{est}	mm	0.283	0.188	0.357	0.164	
v_{21}	*of* F_{est}	mm	0.279	0.179	0.357	0.157	
v_{24}	*of* F_{est}	mm	0.392	0.285	0.530	0.267	
$v_{i,mod}$	$= 4/3\ (v_{04} - v_{01})$	mm	0.398	0.305	0.493	0.272	
v_s	$= v_i - v_{i,mod}$	mm	-0.059	-0.058	-0.064	-0.052	
v_e	$= 2/3\ (v_{14} + v_{24} - v_{11} - v_{21})$	mm	0.136	0.131	0.209	0.124	
$v_{0.6}$	= *slip at 0.6* F_{max}	mm	0.615	0.615	0.564	0.584	
$v_{0.8}$	= *slip at 0.8* F_{max}	mm	0.948	1.025	0.850	1.001	
v_{24}	*of* F_{max}	mm	0.408	0.357	0.448	0.299	
$v_{0.6,mod}$	$= v_{0.6} - v_{24} + v_{i,mod}$	mm	0.605	0.562	0.609	0.558	
$v_{0.8,mod}$	$= v_{0.8} - v_{24} + v_{i,mod}$	mm	0.937	0.973	0.895	0.975	
k_i	$= 0.4F_{est}/v_i$	kN/mm	77.246	105.77	83.809	119.02	
$\boldsymbol{K}_{\mathbf{s,0.4\ SLS}}$	$= 0.4F_{est}/v_{i,mod}$	kN/mm	65.879	85.827	72.978	96.159	80.210
$\boldsymbol{K}_{\mathbf{s,0.6\ ULS}}$	$= 0.6F_{max}/v_{0.6,mod}$	kN/mm	69.310	83.245	64.537	84.328	75.355
$\boldsymbol{K}_{\mathbf{s,0.8\ COL}}$	$= 0.8F_{max}/v_{0.8,mod}$	kN/mm	59.624	64.134	58.616	64.340	61.678

Slip modulus at SLS, $K_{s,0.4} = 0.4F_{est}/v_{i,mod}$ Eq. A5-1

Slip modulus at ULS, $K_{s,0.6} = 0.6F_{max}/v_{0.6,mod}$ Eq. A5-2

Slip modulus at collapse, $K_{s,0.8} = 0.8F_{max}/v_{0.8,mod}$ Eq. A5-3

Initial slip, $v_i = v_{04}$ Eq. A5-4

Modified initial slip, $v_{i,mod}$ $= \frac{4}{3}(v_{04} - v_{01})$ Eq. A5-5

Slip at $0.6F_{max}$ $= v_{06}$ Eq. A5-6

Modified slip at $0.6F_{max}$, $v_{0.6,mod}$ $= v_{0.6} - v_{24} + v_{i,mod}$ Eq. A5-7

Slip at $0.8F_{max}$ $= v_{08}$ Eq. A5-8

Modified slip at $0.8F_{max}$, $v_{0.8,mod}$ $= v_{0.8} - v_{24} + v_{i,mod}$ Eq. A5-9

5.3 Phase 1 connection push-out test

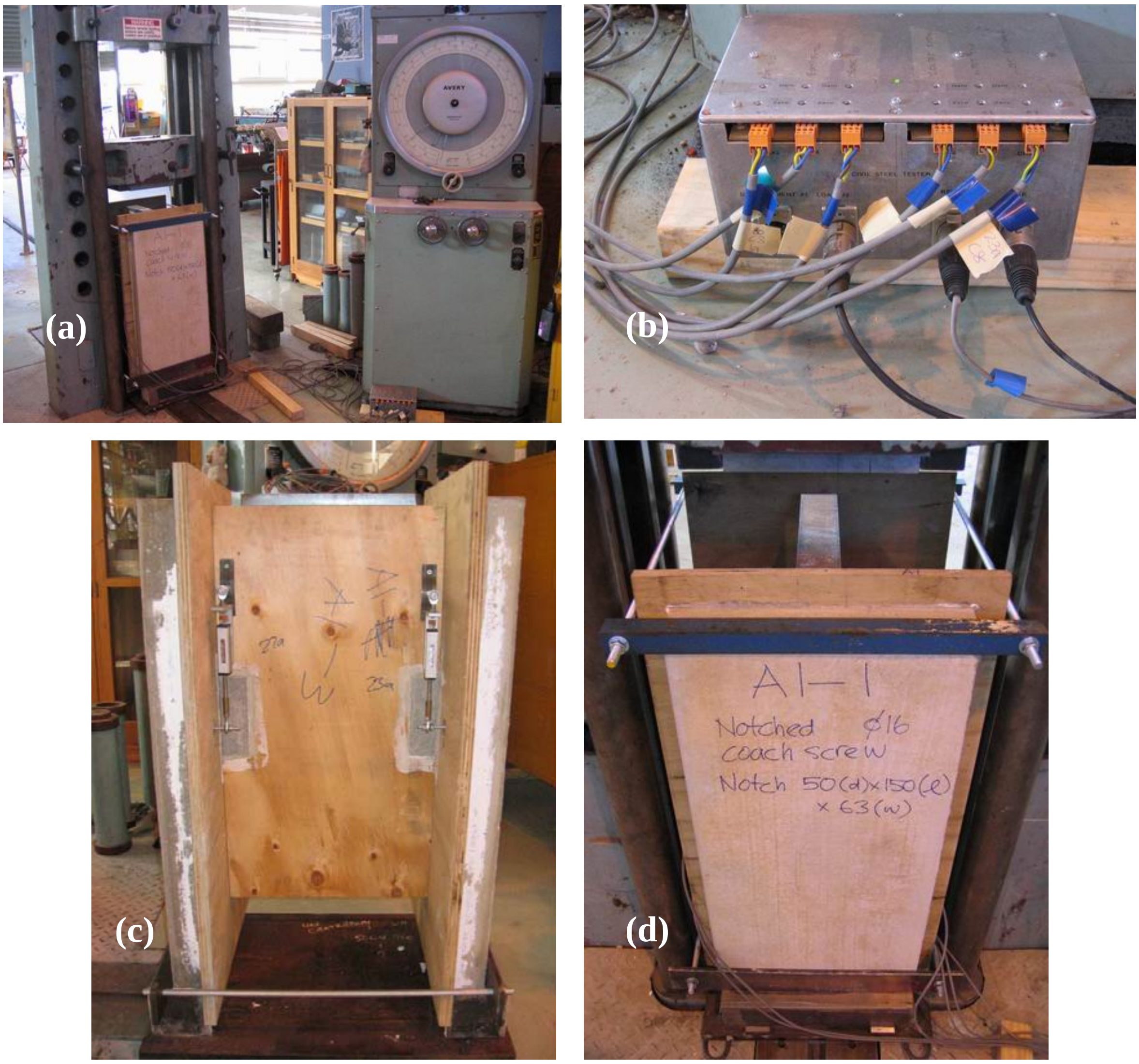


Fig. A5-4. (a) Avery universal testing machine used for connection push-out test; (b) data acquisition box for load and relative slip; (c) Symmetrical push-out specimen set up with potentiometers; and (d) Top and bottom steel plate bracing

Fig. A5-5. Specimen A1 – 150 mm rectangular notched coach screw connection. Shear failure along length of notch. Coach screw in tension (bottom left)

Fig. A5-6. Specimen H1 – Double LVL 150 mm rectangular notched coach screw connection. Shear failure along length of notch.

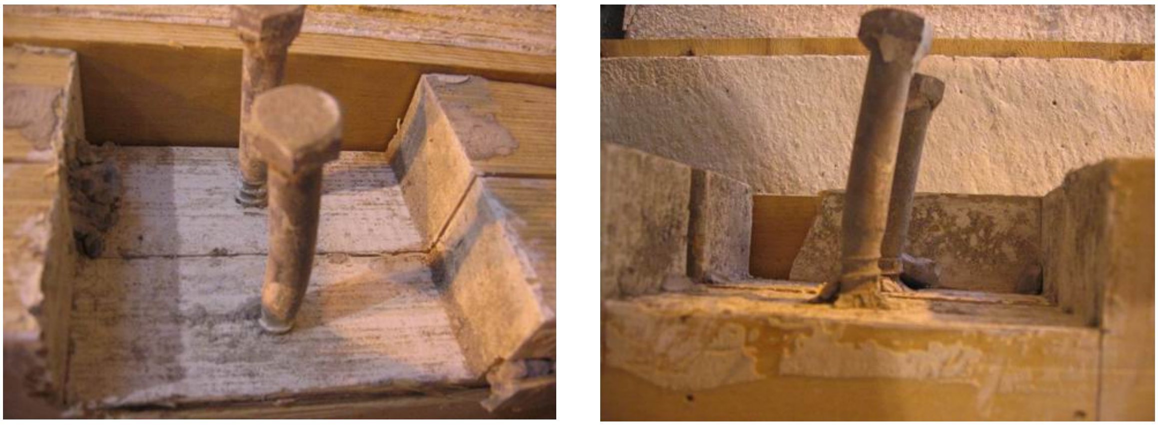


Fig. A5-7. Specimen H1 – Double LVL 150 mm rectangular notched coach screw connection. Inside coach screws in tension

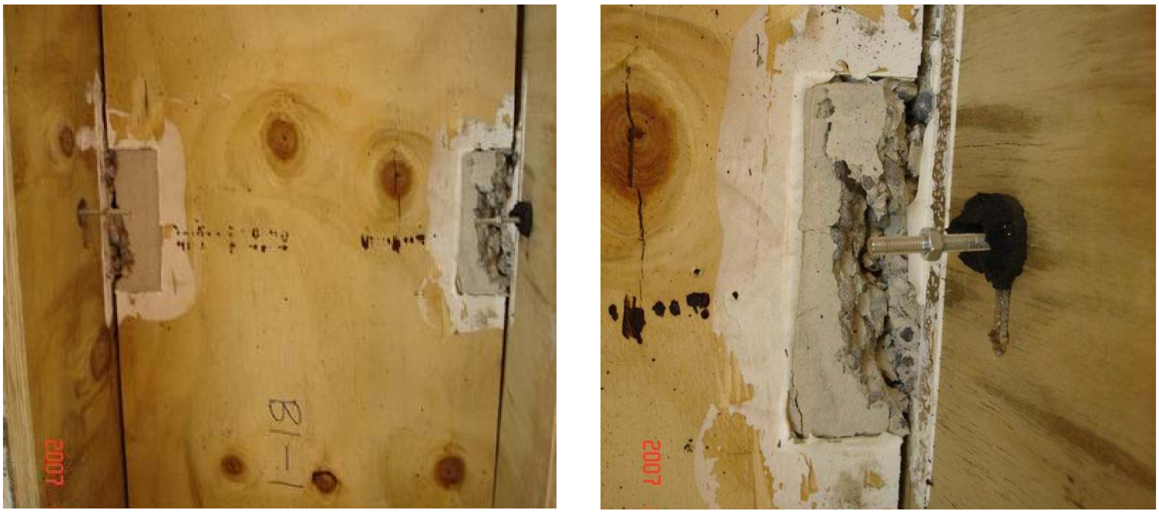


Fig. A5-8. Specimen B1 – 150 mm rectangular notch without coach screw. Complete shear off along length of notch

Fig. A5-9. Specimen D1 – Dovetail notch without coach screw. Complete shear off along length of notch

Fig. A5-10. Specimen H3 – Double tooth metal plate. Complete plate tear off failure

5.4 Phase 2 connection push-out test

Fig. A5-11. Specimen R – 300 mm rectangular notched coach screw. Shear failure along notch length

Fig. A5-12. Specimen T – Triangular notched coach screw connection. Shear failure along notch length and coach screw in tension

Fig. A5-13. Specimen P – Toothed metal plate connection. Plate tear off failure embedded inside

APPENDIX

6. Construction of beams for short-term collapse test

This appendix presents the construction process of beams for short-term test in the form of photographs. 5 beams were built outdoor (outside the laboratory) and 8 beams built indoor (Structures Laboratory). Table A6-1 summarizes the beams that have been constructed.

Table A6-1. Beams constructed indoor and outdoor

Location	Beams
Indoor	A1, A2, B1, B2, E1, E2, G1
Outdoor	C1, D1, D2, F1, F2

Note: Beams D2 and F2 were tested under cyclic load. This test is beyond the scope of this thesis.

6.1 Construction of beams indoor

Fig. A6-1. Notch cutting using a jig-saw

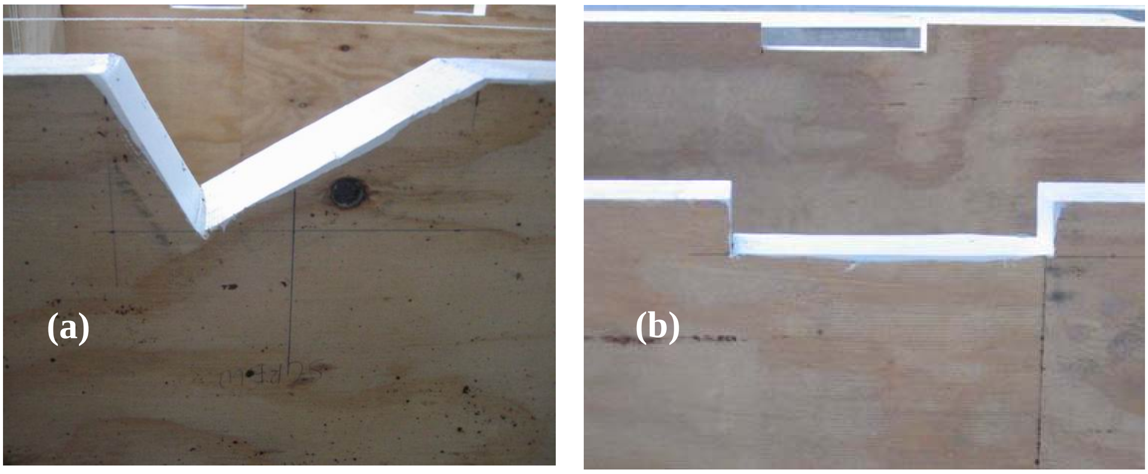


Fig. A6-2. (a) Triangular notched cut in beam C2; (b) 150 mm rectangular notched cut in beams B1 and B2 (in the background)

Fig. A6-3. Setting up of indoor beams using thread line for correct alignment

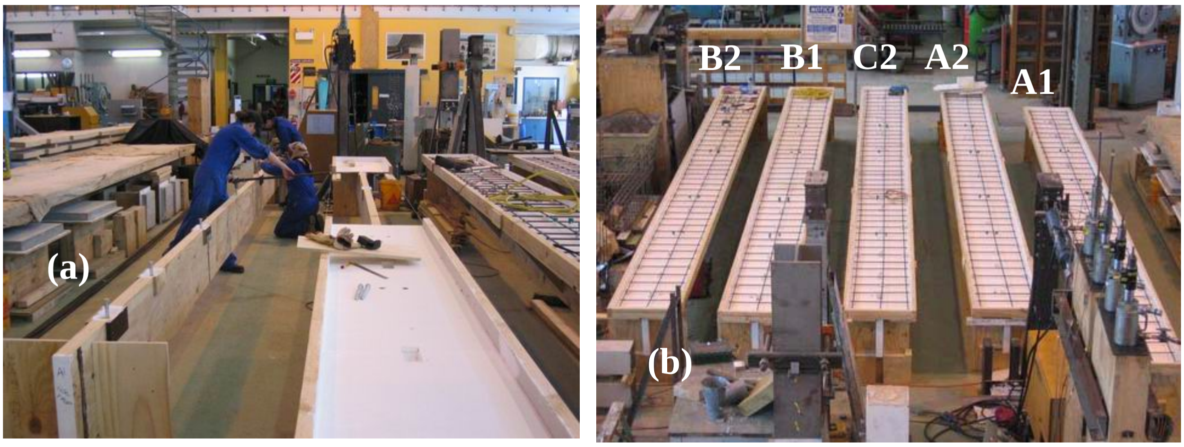


Fig. A6-4. (a) Constructing formwork for the flange; (b) Beams A1, A2, B1, B2 and C2 ready for concreting in the laboratory

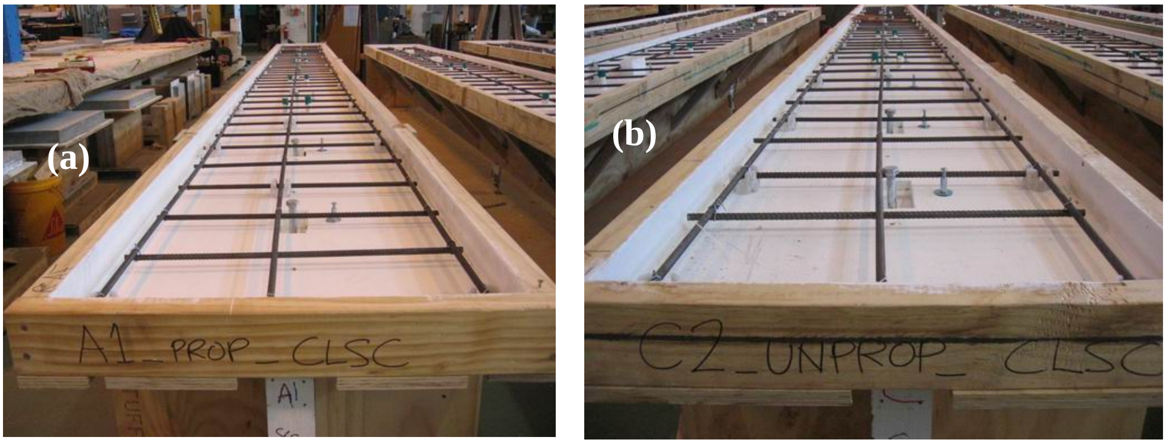


Fig. A6-5. Close up of beams that were ready for concreting: (a) beam A1; (b) beam C2

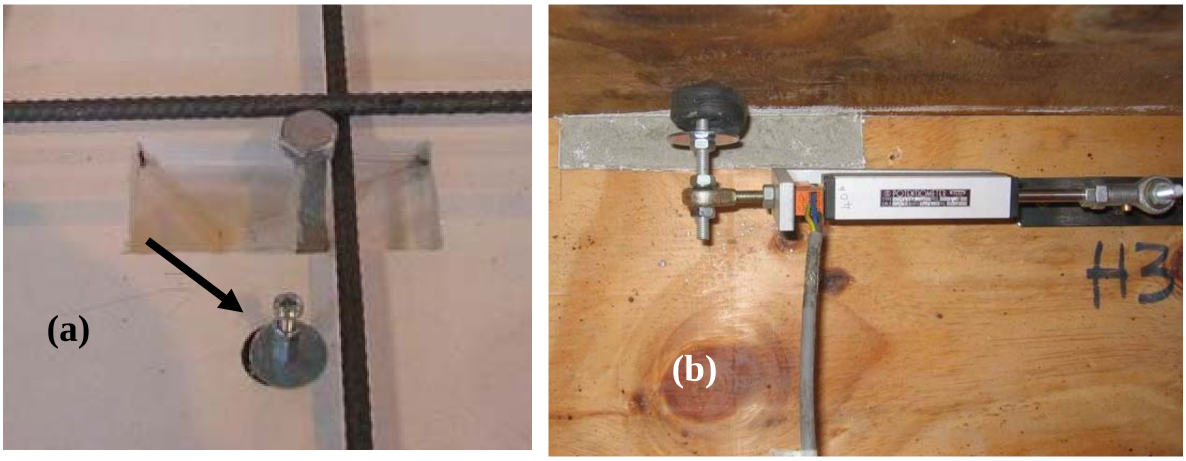


Fig. A6-6. (a) 6 mm threaded rod, screw and washer mounting concreted adjacent to connection for horizontal relative slip measurement using potentiometer; (b) potentiometer attached to measure horizontal relative slip

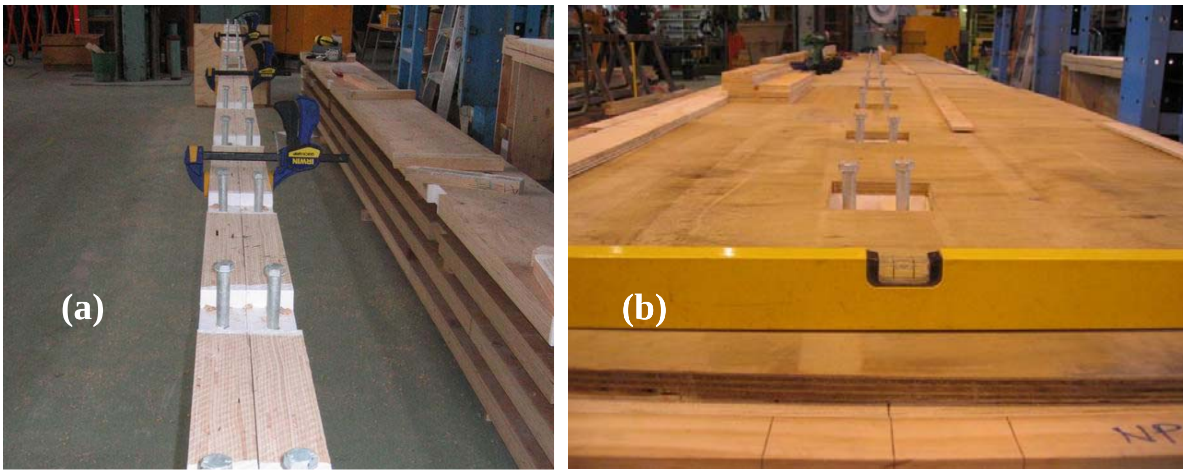


Fig. A6-7. Construction of beam G1: (a) double 150 mm rectangular notch with coach screw; (b) formwork leveling

Fig. A6-8. Concreting of beam G1

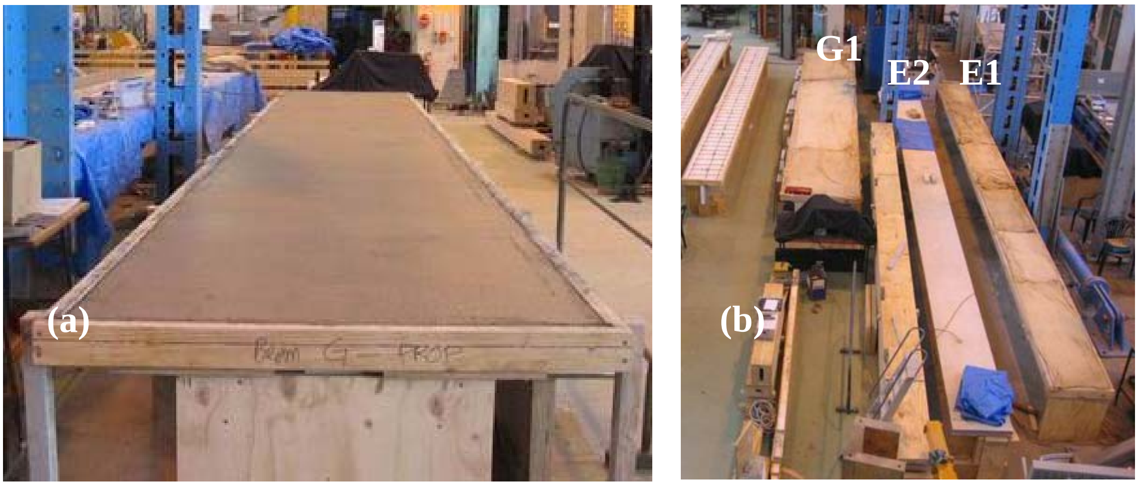


Fig. A6-9. (a) Beam G1 after concreting; (b) beams G1, E1 and E2

Fig. A6-10. Propping of beams at mid-span

Fig. A6-11. Beam A2 with notched pocketed and grouted after 7 days of concreting

Fig. A6-12. Four point lifting of beam using a spreader bar

Fig. A6-13. Notches visible under side of beams, triangular notch (left) and rectangular notch (right)

6.2 Construction of beams outdoor

Fig. A6-14. Setting out of beam C1 with a triangular notched coach screw connection

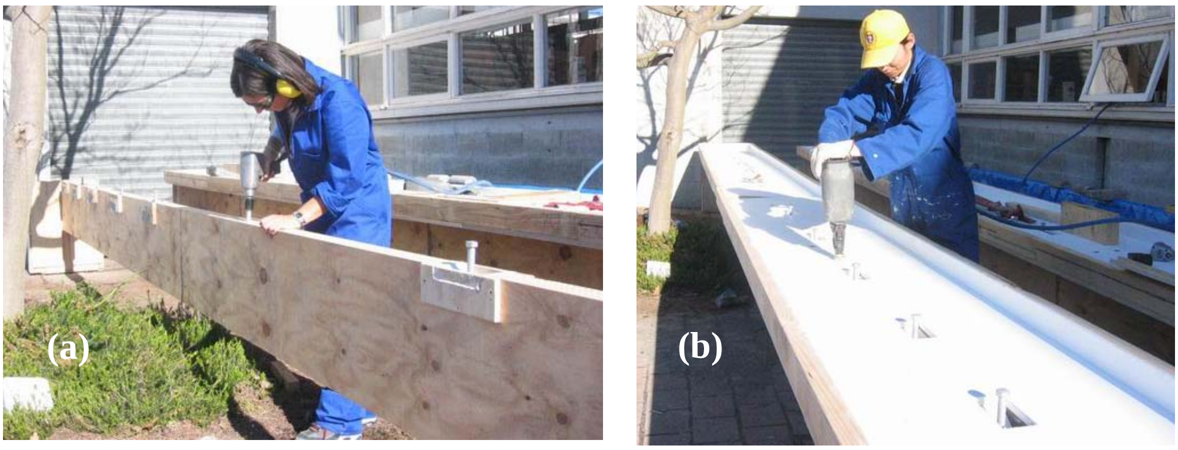


Fig. A6-15. (a) Inserting coach screws using a pneumatic powered socket gun; (b) Attaching flange formwork to the LVL joist with a nail-gun

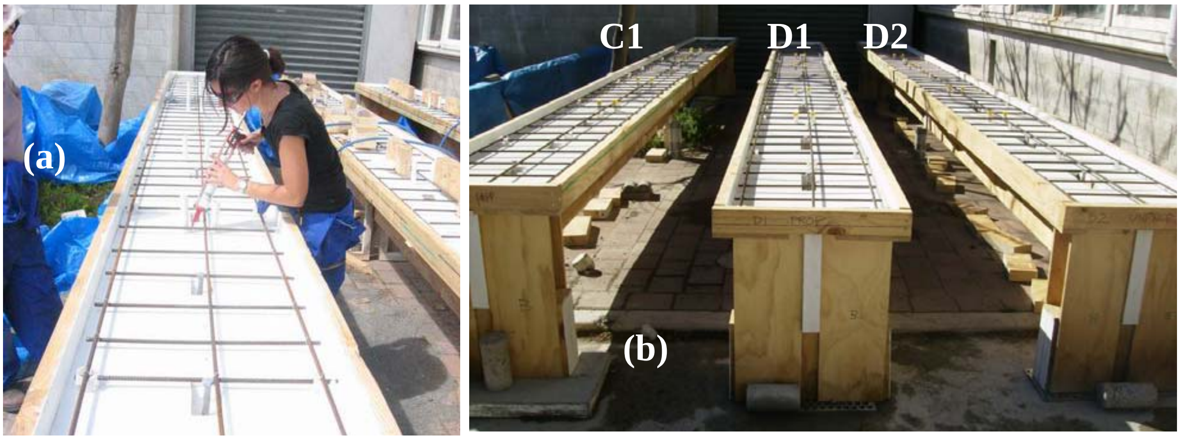


Fig. A6-16. (a) Sealing holes in the formwork; (b) Beams C1, D1 and D2 ready for concreting

Fig. A6-17. (a) Concreting of beams C2, D1 and D2; (b) after concreting

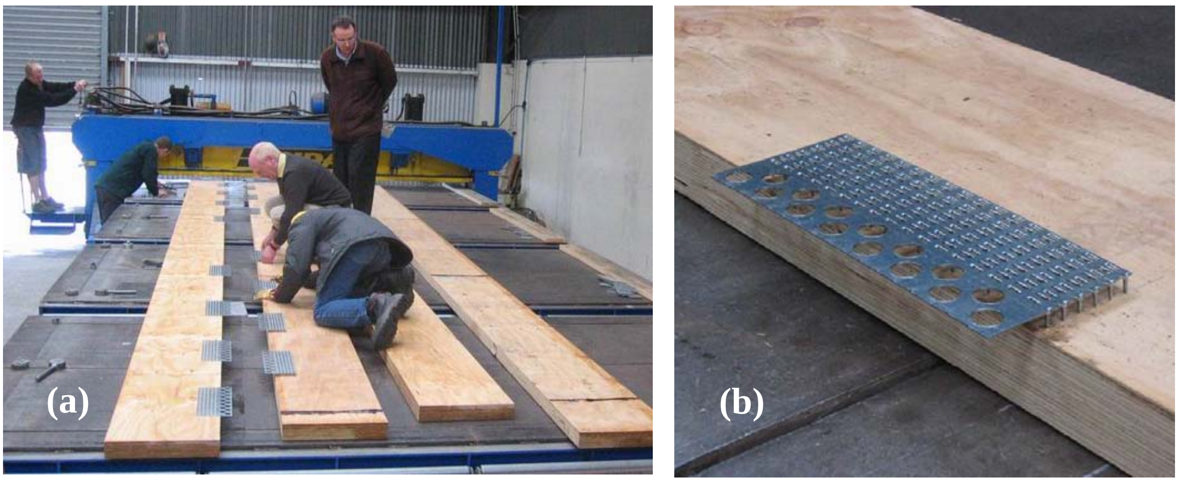


Fig. A6-18. (a) Pressing in of metal plates into LVL for beams F1 and F2 at Westlake Timbers, Christchurch (a fabricator of Mitek); (b) Close up of metal plate

Fig. A6-19. Setting out of beams F1 (left) and F2 (right)

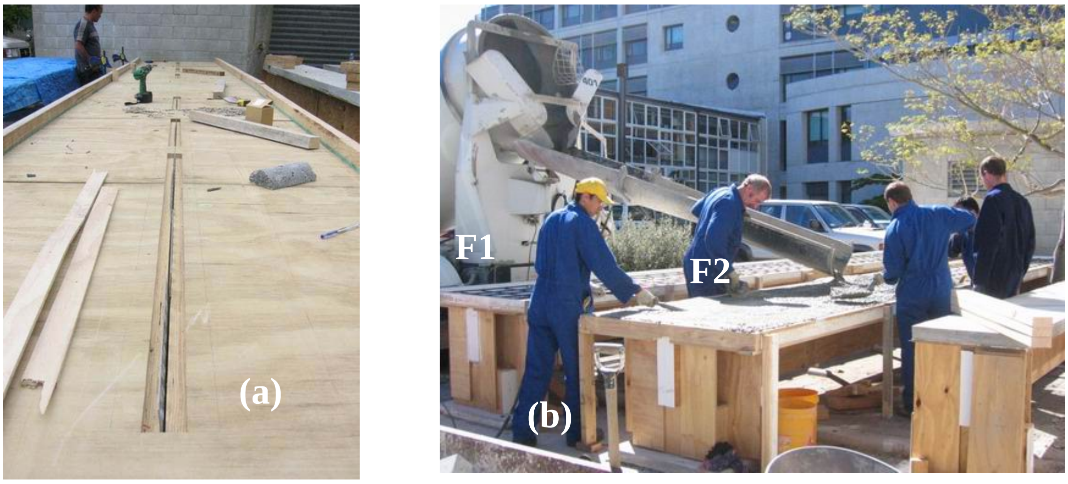


Fig. A6-20. (a) Construction of formwork; (b) concreting beams F1 and F2

Fig. A6-21. (a) Outside beams; (b) lifting of outside beams into the laboratory for testing

APPENDIX

7. Short-term beam collapse test

This appendix presents photographs and load-deflection graphs of beams tested to collapse.

7.1 Experimental setting up

Fig. A7-1. Four point bending test set up: (a) Overall view; (b) Loading spreader beam; (c) 400 kN loading ram

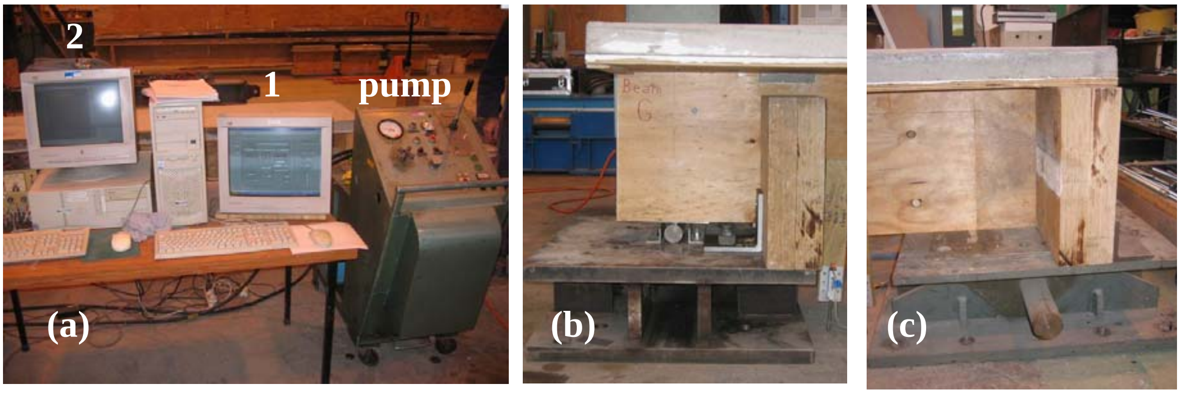


Fig. A7-2. (a) Computer 1 to control load regime, computer 2 to acquire data, and hydraulic pump to apply load; (b) roller support; (c) pin support

Fig. A7-3. Potentiometers used to measure connection relative slips and mid-span displacement

7.2 Failures in beams

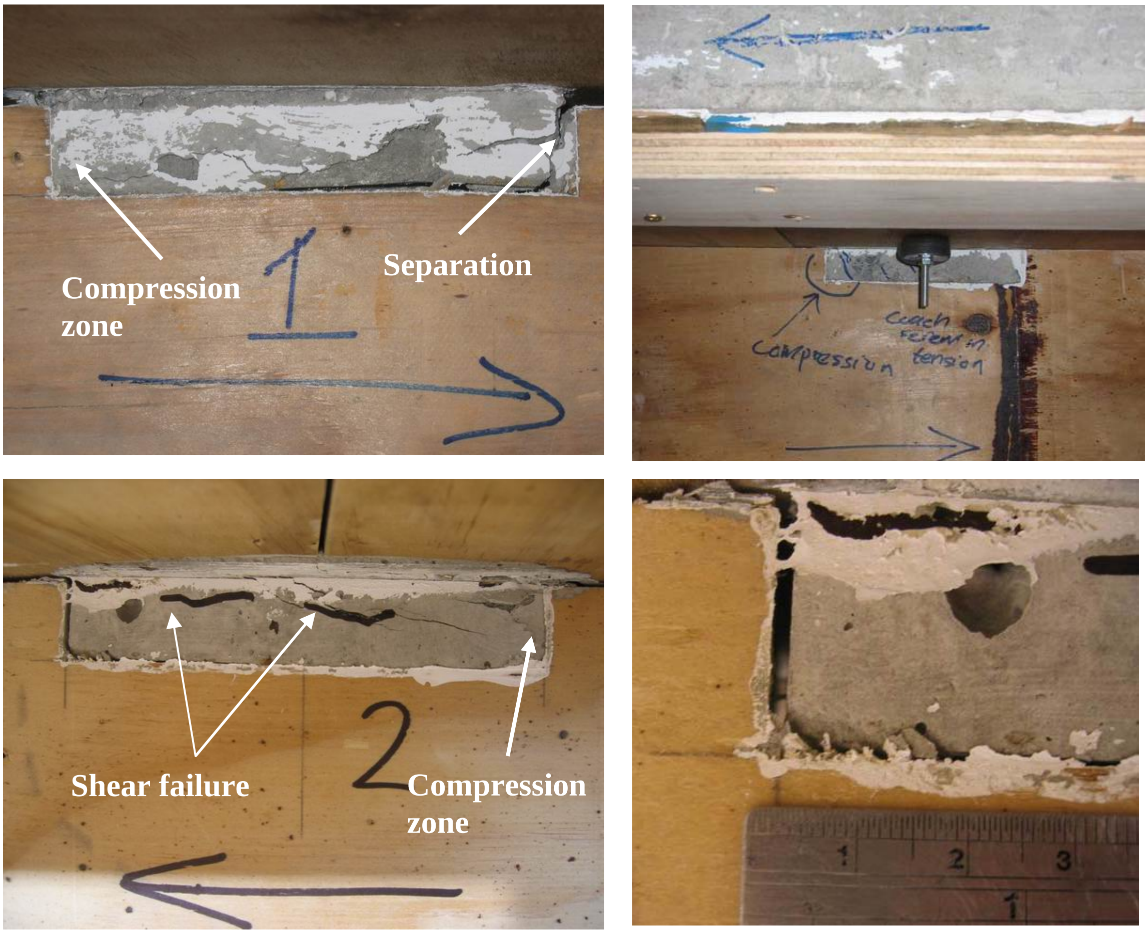


Fig. A7-4. Shear failure along length of 150 mm rectangular notch with coach screw in beam A1. 3 to 4 mm separation of notch from LVL (bottom right).

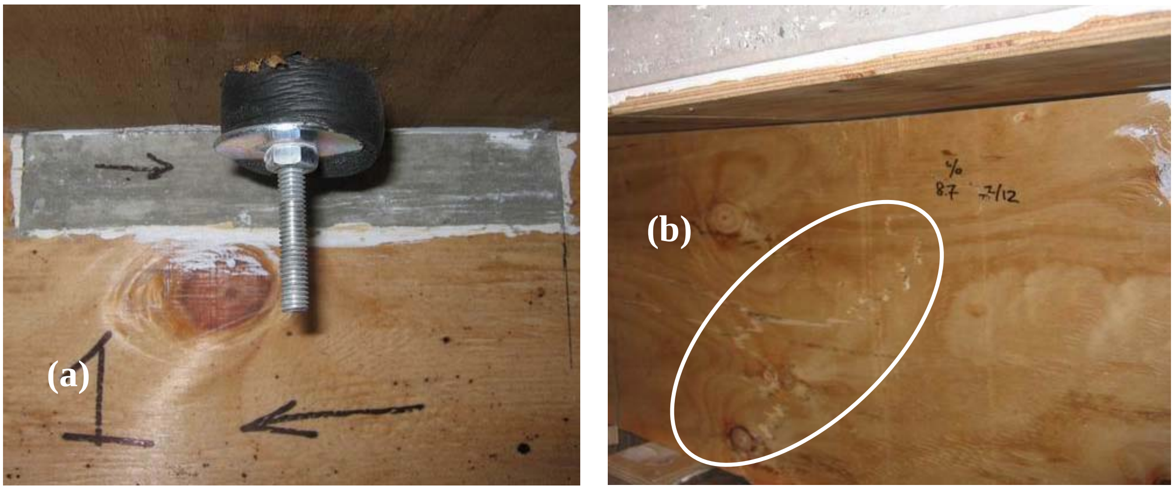


Fig. A7-5. Beam B1: (a) no sign of failure in connections; (b) sudden bending tension failure in LVL at one third span

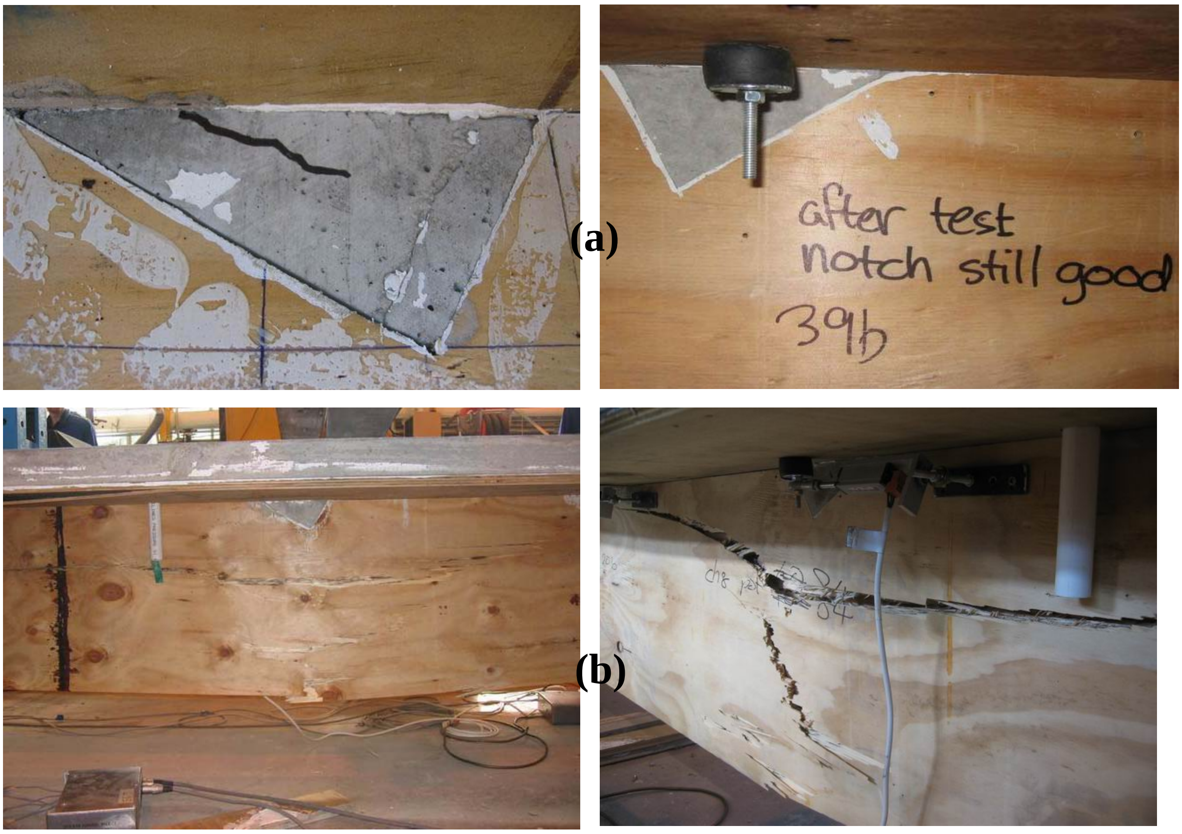


Fig. A7-6. Beam C2: (a) no obvious failure in connections apart from hair line cracks and separation from LVL in one notch; (b) sudden bending tension failure in LVL at one third span

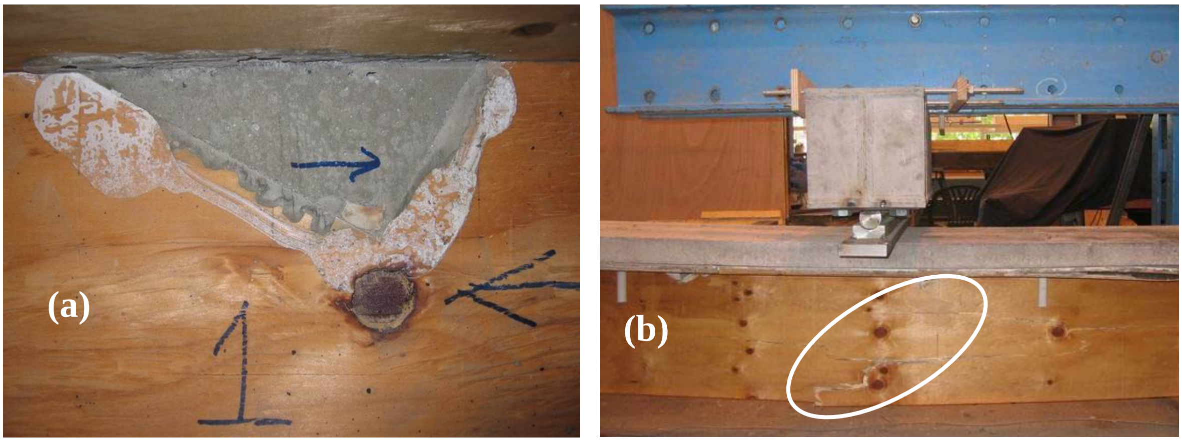


Fig. A7-7. Beam C1: (a) no sign of failure in connections; (b) sudden bending tension failure in LVL at one third span

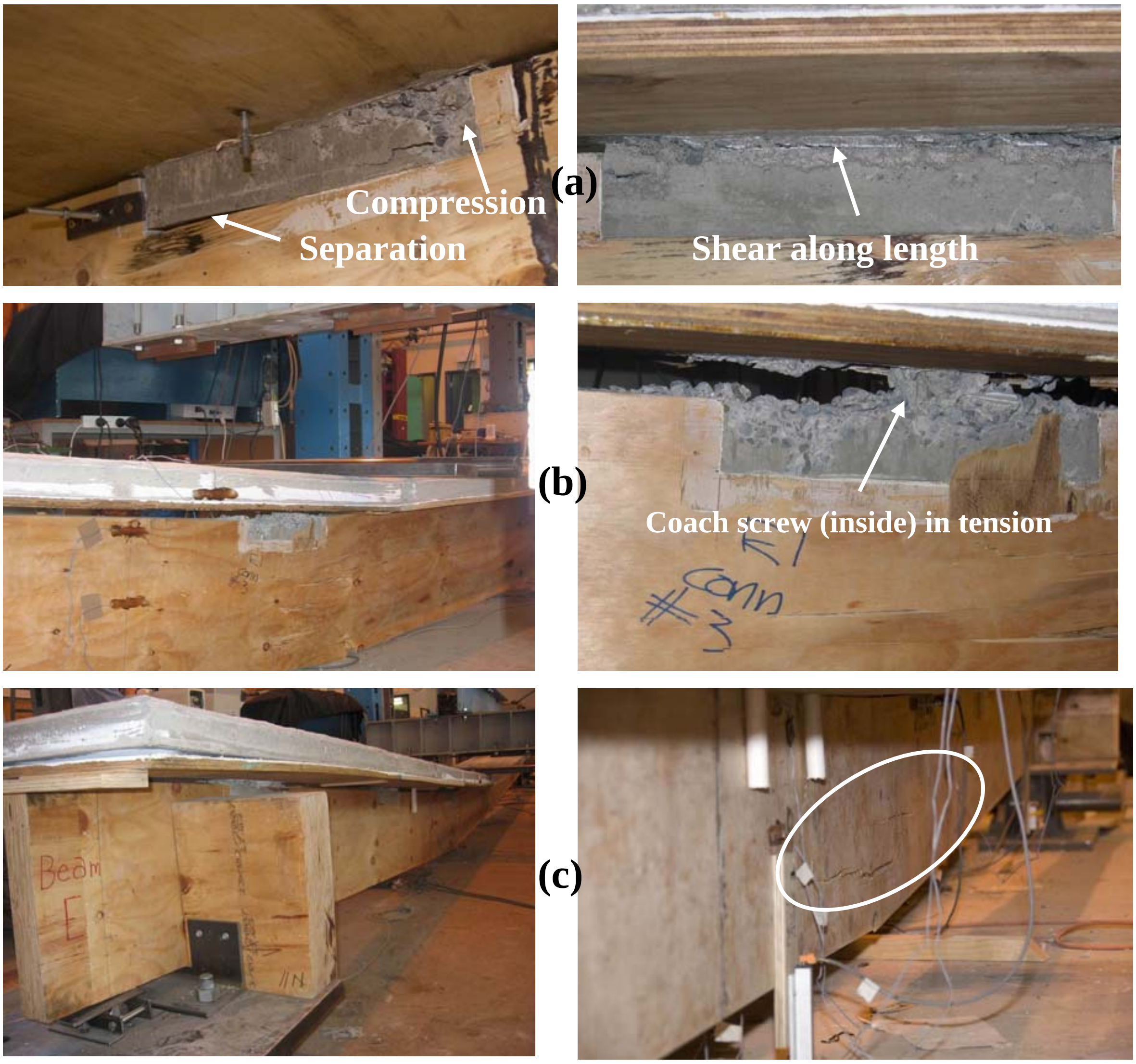


Fig. A7-8. Beam E1: (a) Shear along notch length and compression failure; (b) complete connection failure with coach screw in tension at one third span; (c) LVL brittle tension at one third span

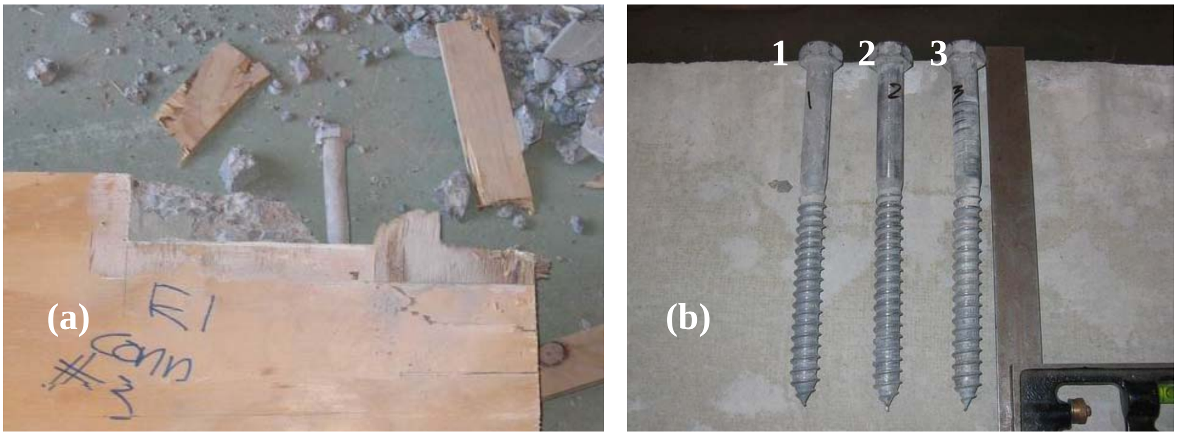
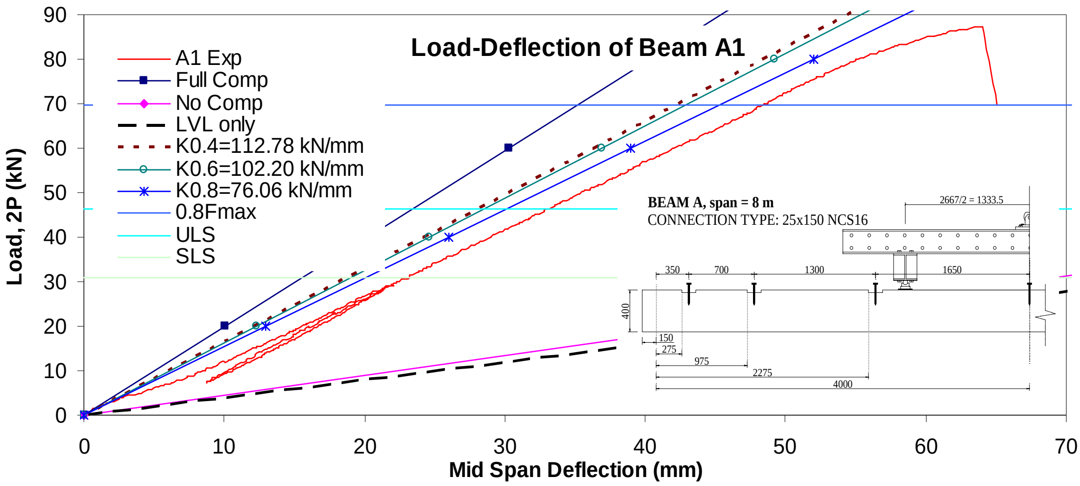


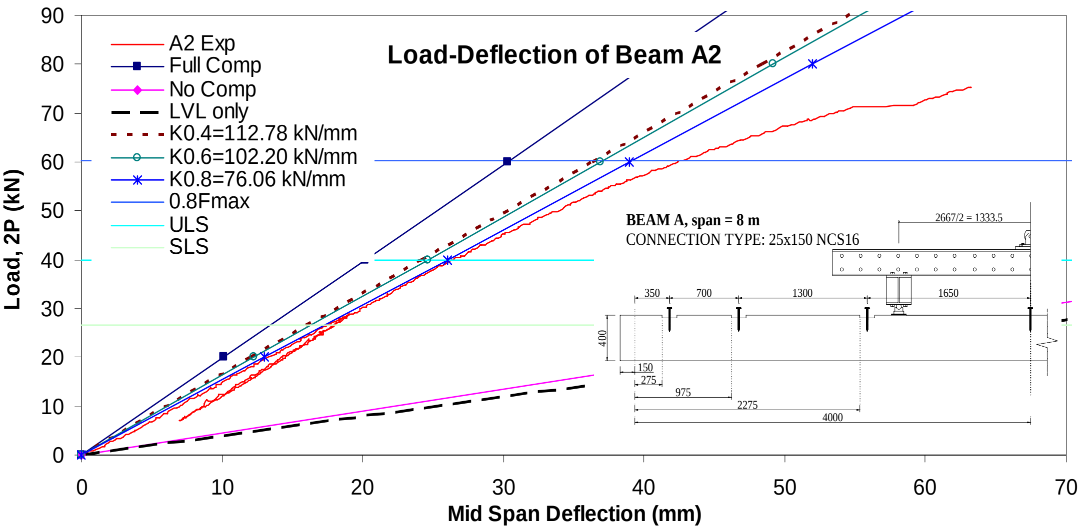
Fig. A7-9. (a) Dismantling of beam E1 after test; (b) Coach screws 1 (nearest to support), 2, 3 (nearest to point load at one third span). Only coach screw 3 had slight bent.

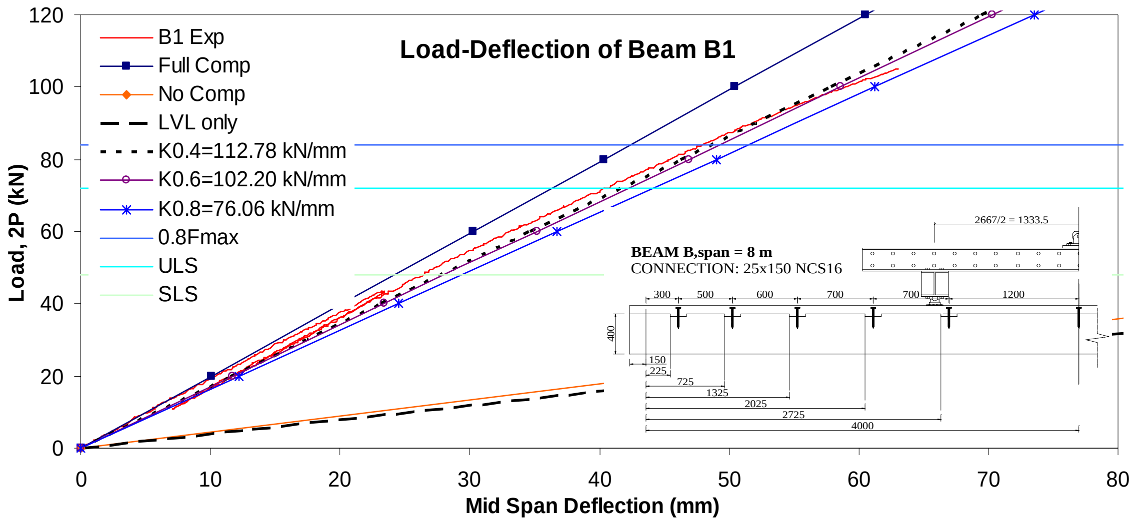
Fig. A7-10. Beam G1: No failure in connection (top left), and LVL bending tension failure at one third span

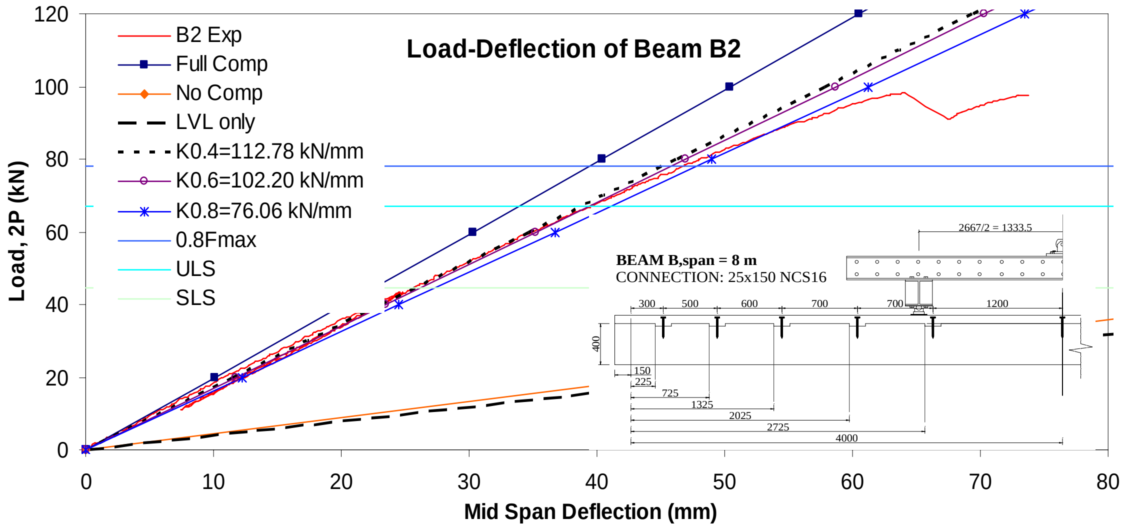
Fig. A7-11. Beam F1: sudden bending tension failure in LVL at one third span

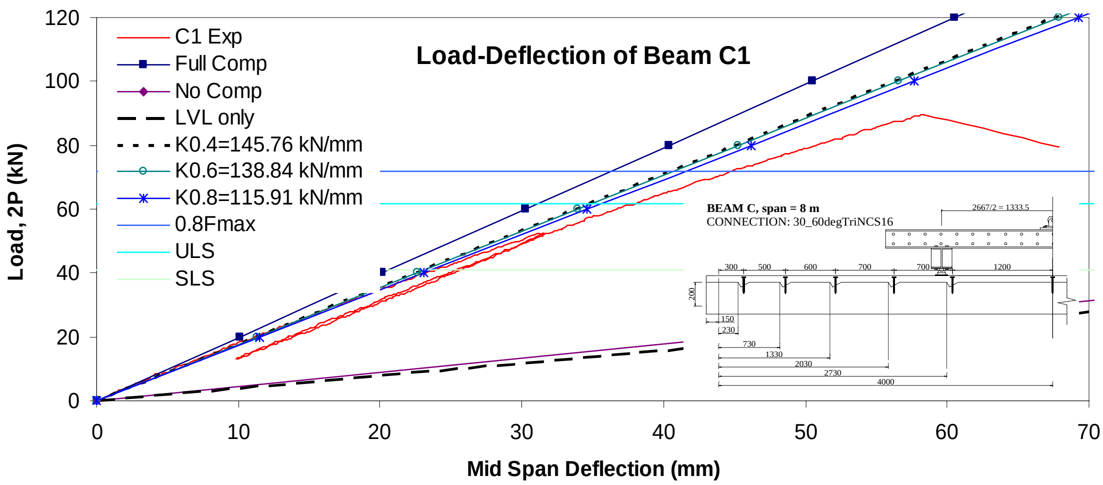
7.3 Load-deflection graphs of tested beams

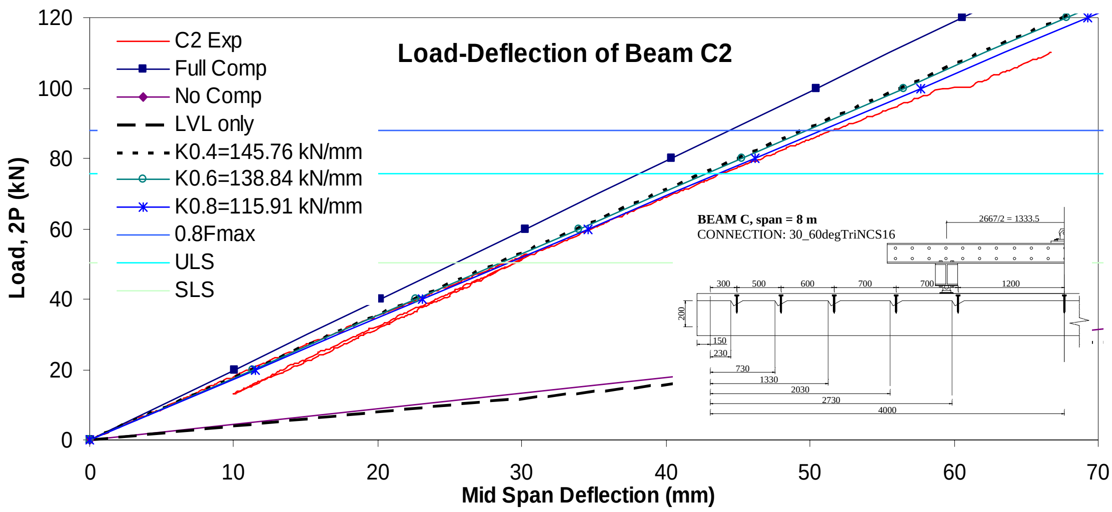


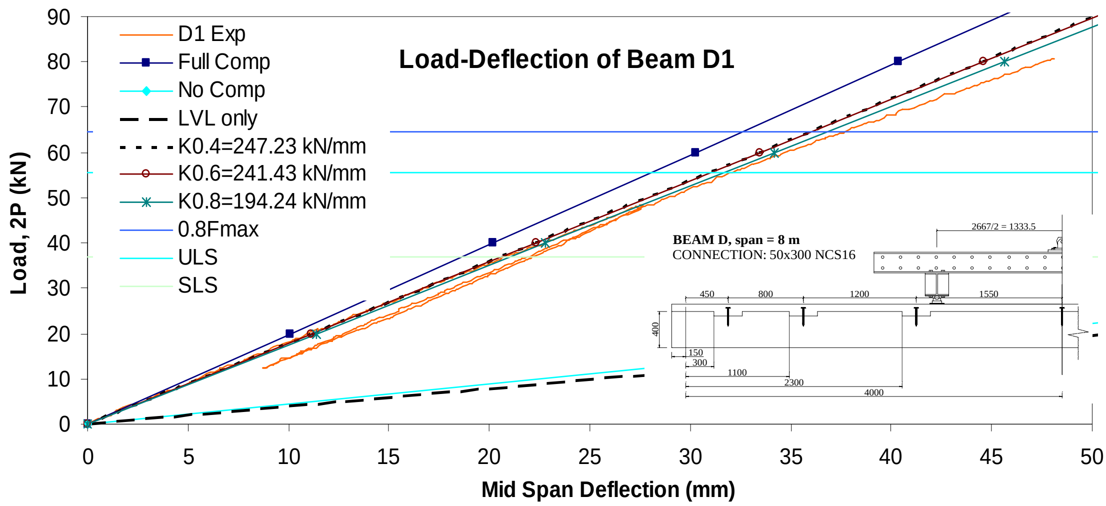


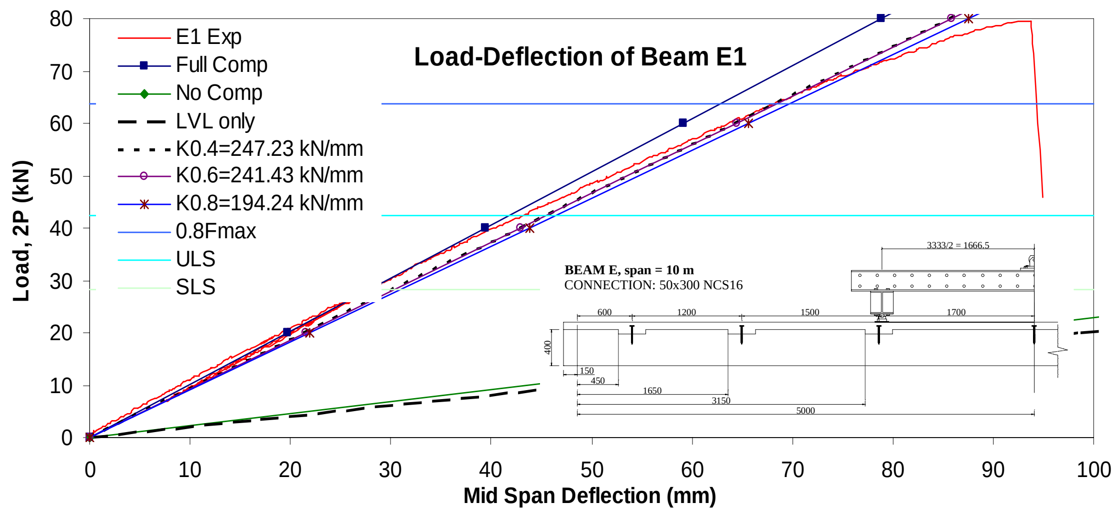

Load-Deflection of Beam B1
B1 Exp
Full Comp
No Comp
LVL only
K0.4=112.78 kN/mm
K0.6=102.20 kN/mm
K0.8=76.06 kN/mm
0.8Fmax
ULS
SLS
BEAM B,span = 8 m
CONNECTION: 25x150 NCS16
2667/2 = 1333.5
Load, 2P (kN)
Mid Span Deflection (mm)

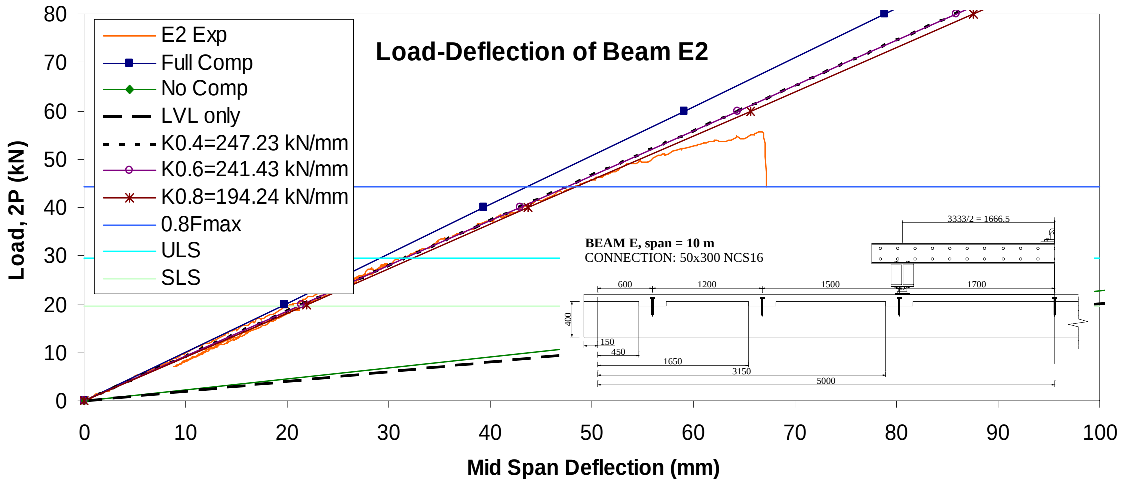

Load-Deflection of Beam B2
B2 Exp
Full Comp
No Comp
LVL only
K0.4=112.78 kN/mm
K0.6=102.20 kN/mm
K0.8=76.06 kN/mm
0.8Fmax
ULS
SLS
BEAM B,span = 8 m
CONNECTION: 25x150 NCS16
2667/2 = 1333.5
Load, 2P (kN)
Mid Span Deflection (mm)

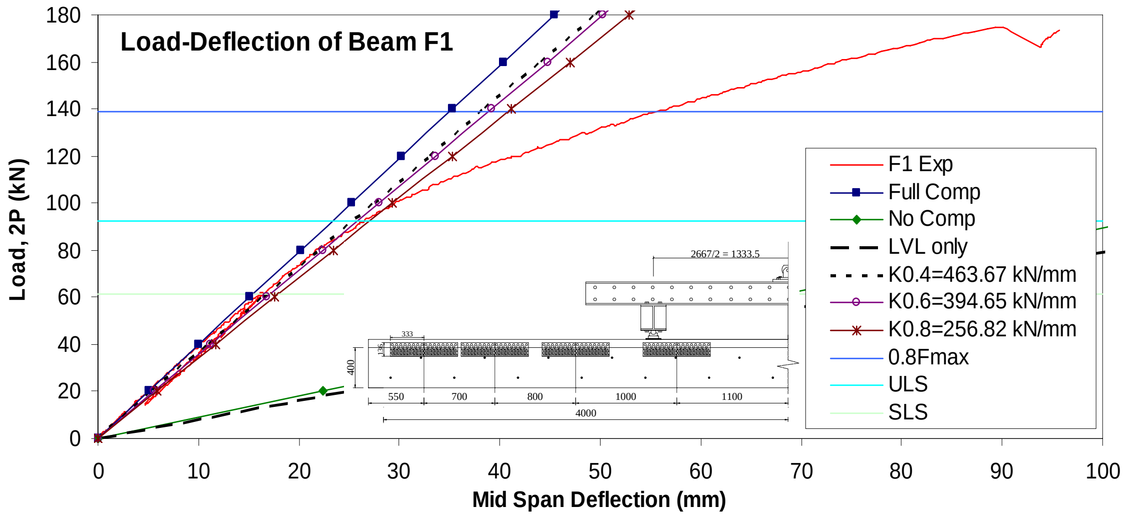

Load-Deflection of Beam C1
C1 Exp
Full Comp
No Comp
LVL only
K0.4=145.76 kN/mm
K0.6=138.84 kN/mm
K0.8=115.91 kN/mm
0.8Fmax
ULS
SLS
BEAM C, span = 8 m
CONNECTION: 30_60degTriNCS16
Load, 2P (kN)
Mid Span Deflection (mm)

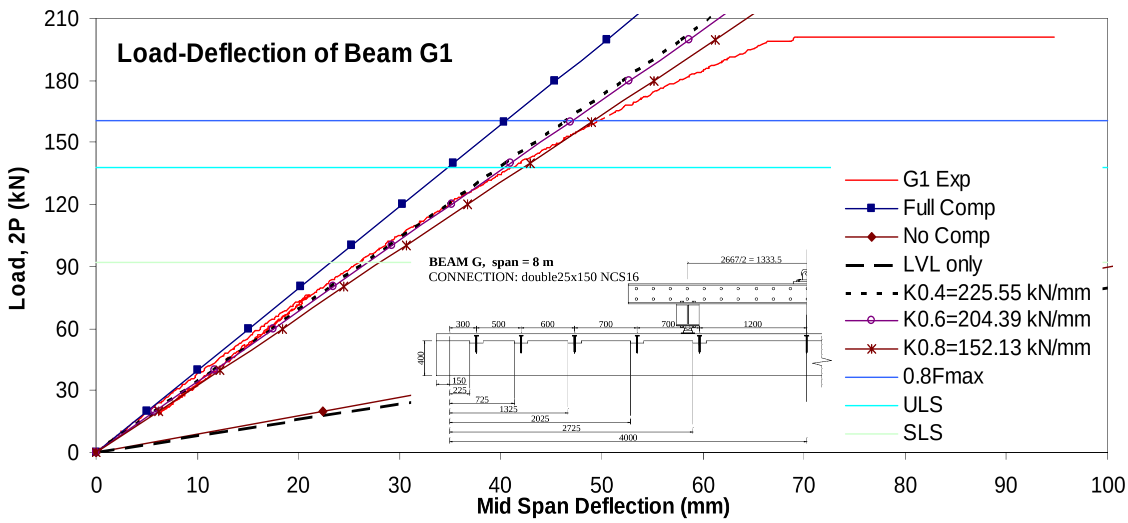

Load-Deflection of Beam C2
C2 Exp
Full Comp
No Comp
LVL only
K0.4=145.76 kN/mm
K0.6=138.84 kN/mm
K0.8=115.91 kN/mm
0.8Fmax
ULS
SLS
BEAM C, span = 8 m
CONNECTION: 30_60degTriNCS16
Load, 2P (kN)
Mid Span Deflection (mm)


Load-Deflection of Beam D1
D1 Exp
Full Comp
No Comp
LVL only
K0.4=247.23 kN/mm
K0.6=241.43 kN/mm
K0.8=194.24 kN/mm
0.8Fmax
ULS
SLS
BEAM D, span = 8 m
CONNECTION: 50x300 NCS16
Load, 2P (kN)
Mid Span Deflection (mm)


Load-Deflection of Beam E1
E1 Exp
Full Comp
No Comp
LVL only
K0.4=247.23 kN/mm
K0.6=241.43 kN/mm
K0.8=194.24 kN/mm
0.8Fmax
ULS
SLS
BEAM E, span = 10 m
CONNECTION: 50x300 NCS16
Load, 2P (kN)
Mid Span Deflection (mm)

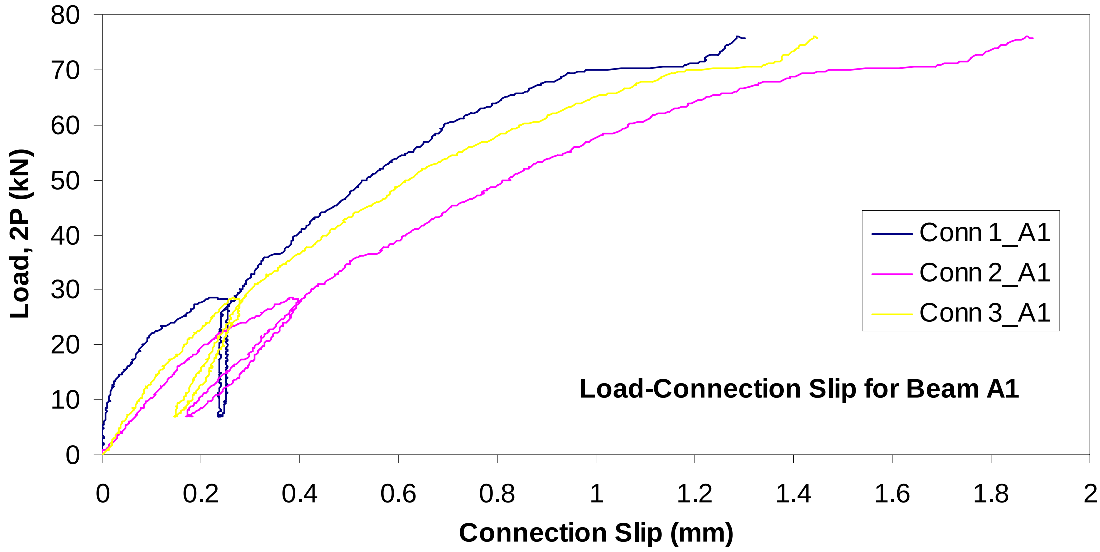

Load-Deflection of Beam E2
Load, 2P (kN)
Mid Span Deflection (mm)
E2 Exp
Full Comp
No Comp
LVL only
K0.4=247.23 kN/mm
K0.6=241.43 kN/mm
K0.8=194.24 kN/mm
0.8Fmax
ULS
SLS
BEAM E, span = 10 m
CONNECTION: 50x300 NCS16

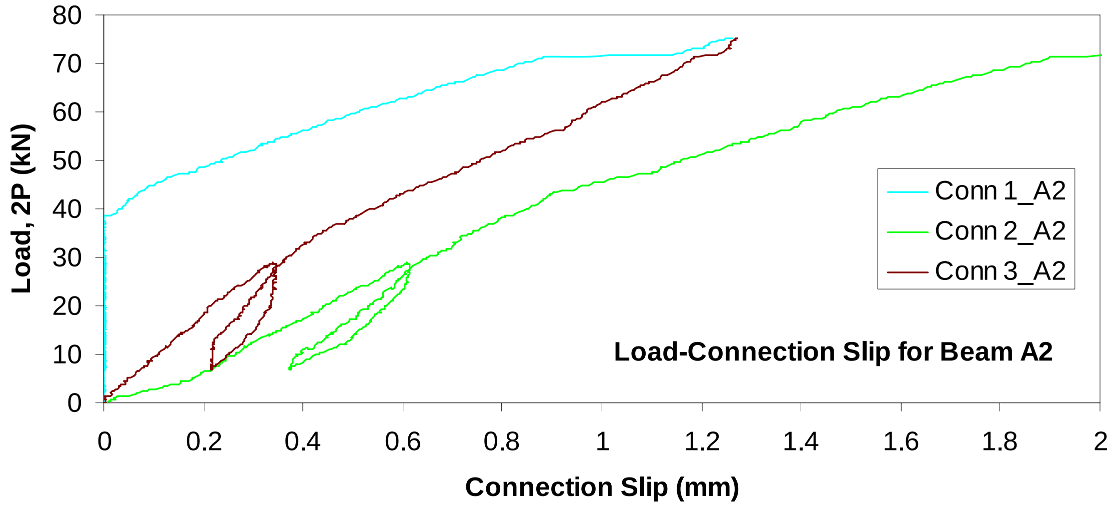

Load-Deflection of Beam F1
Load, 2P (kN)
Mid Span Deflection (mm)
F1 Exp
Full Comp
No Comp
LVL only
K0.4=463.67 kN/mm
K0.6=394.65 kN/mm
K0.8=256.82 kN/mm
0.8Fmax
ULS
SLS

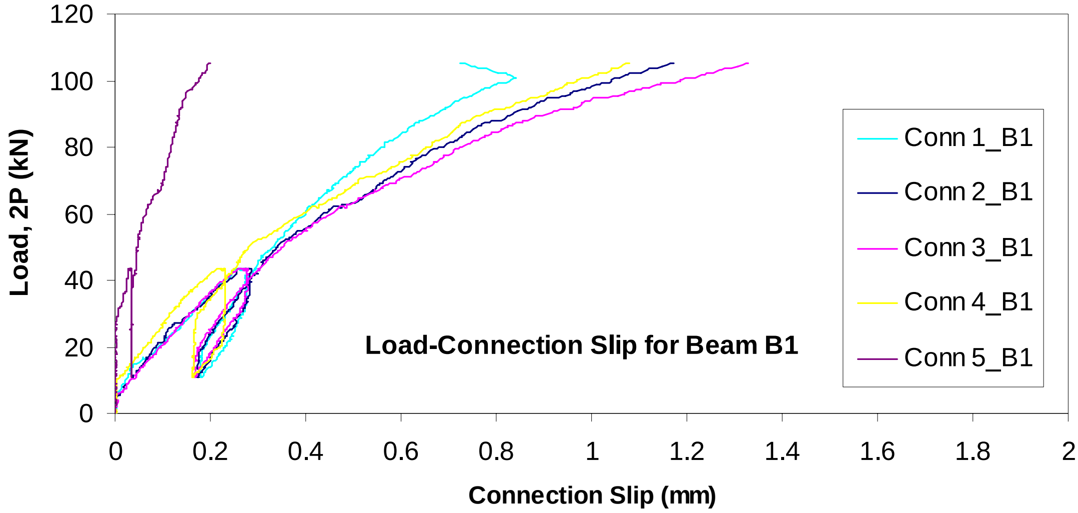

Load-Deflection of Beam G1
Load, 2P (kN)
Mid Span Deflection (mm)
G1 Exp
Full Comp
No Comp
LVL only
K0.4=225.55 kN/mm
K0.6=204.39 kN/mm
K0.8=152.13 kN/mm
0.8Fmax
ULS
SLS
BEAM G, span = 8 m
CONNECTION: double25x150 NCS16

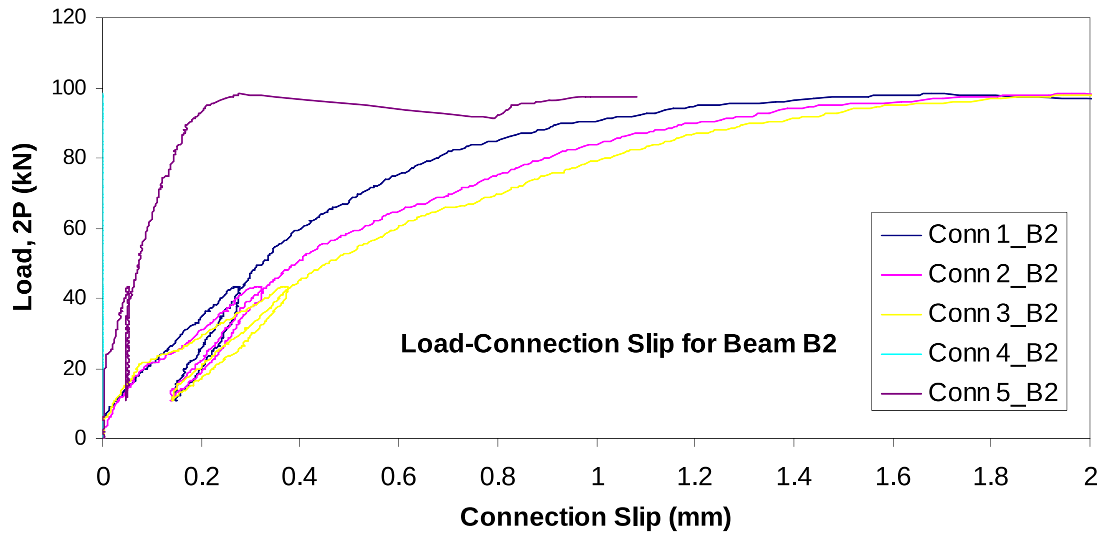
7.4 Connection slips measured for selected beams

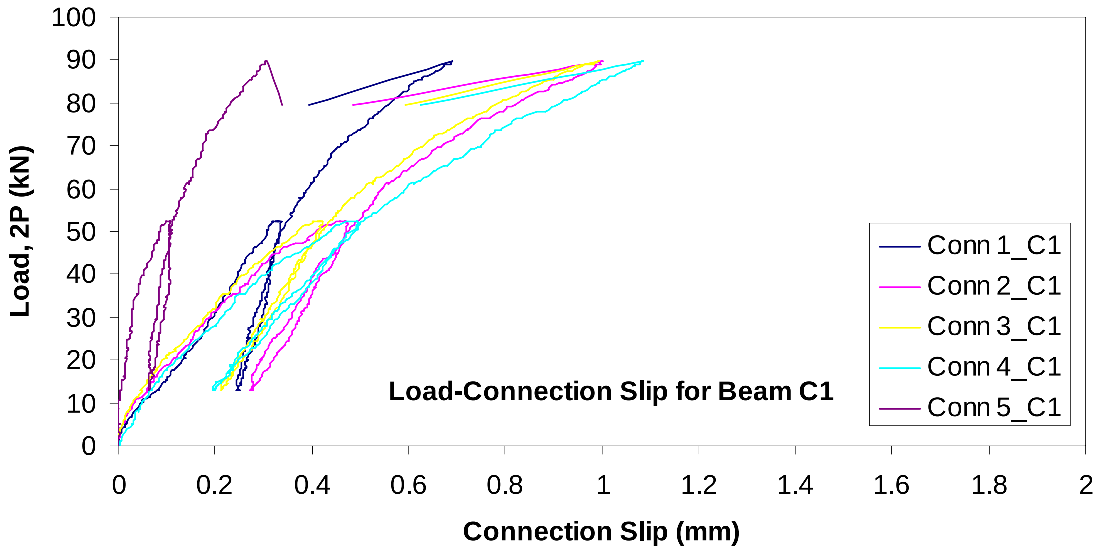
Note that connection number 1 referred to one nearest to the support and connection with the largest number referred to one nearest to the midspan.

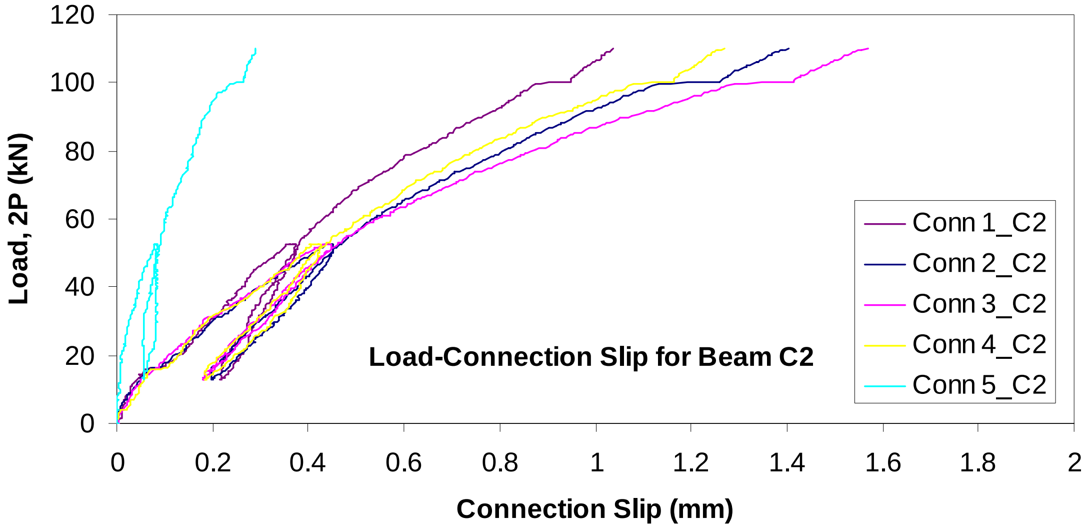


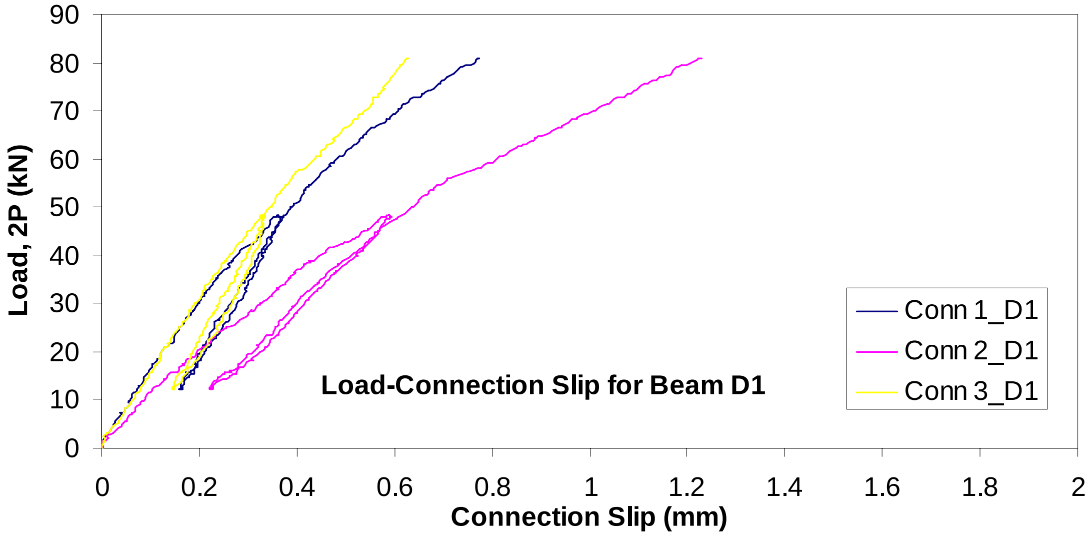


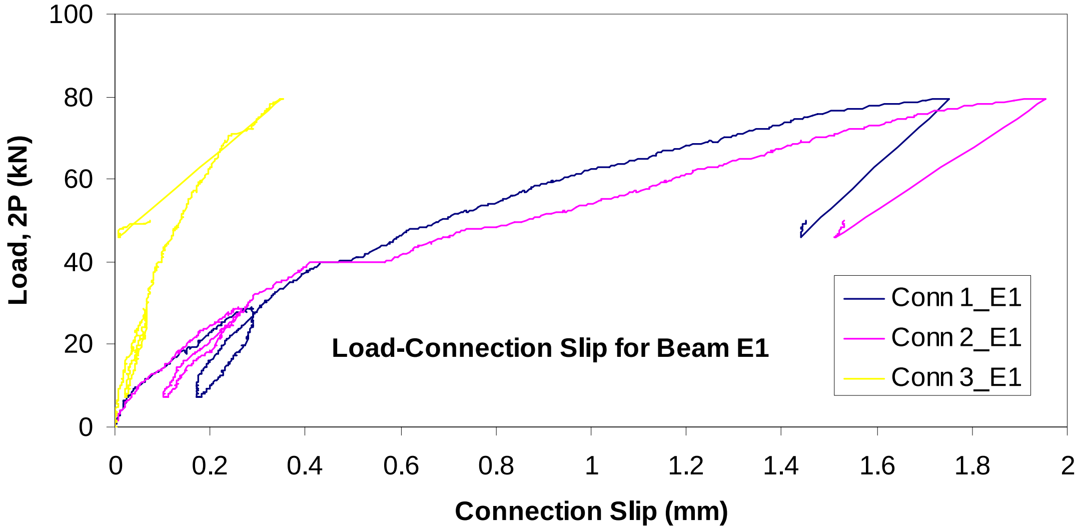

Load-Connection Slip for Beam B1
Load, 2P (kN)
Connection Slip (mm)
Conn 1_B1
Conn 2_B1
Conn 3_B1
Conn 4_B1
Conn 5_B1
0
20
40
60
80
100
120
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2

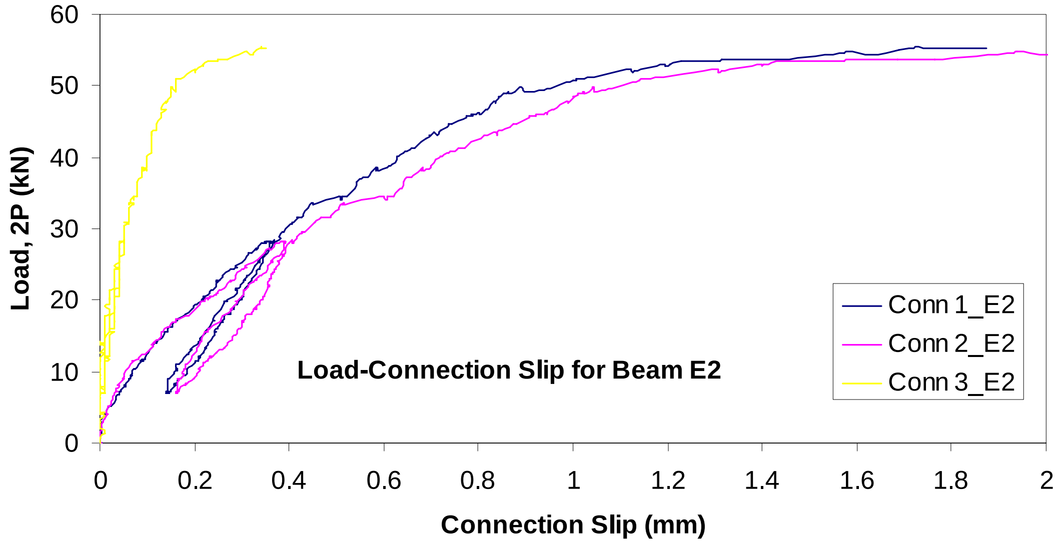

Load-Connection Slip for Beam B2
Load, 2P (kN)
Connection Slip (mm)
Conn 1_B2
Conn 2_B2
Conn 3_B2
Conn 4_B2
Conn 5_B2
0
20
40
60
80
100
120
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2

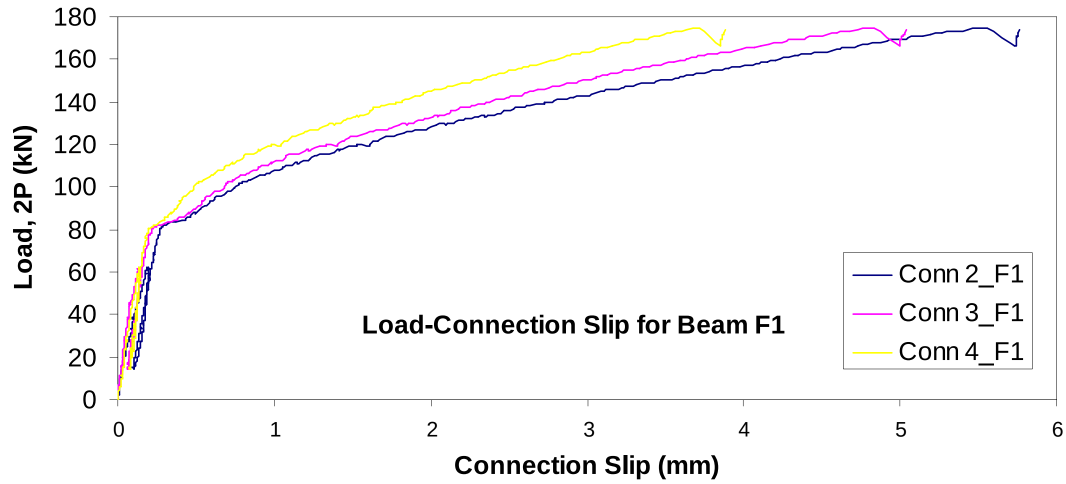

Load-Connection Slip for Beam C1
Load, 2P (kN)
Connection Slip (mm)
Conn 1_C1
Conn 2_C1
Conn 3_C1
Conn 4_C1
Conn 5_C1
0
10
20
30
40
50
60
70
80
90
100
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2

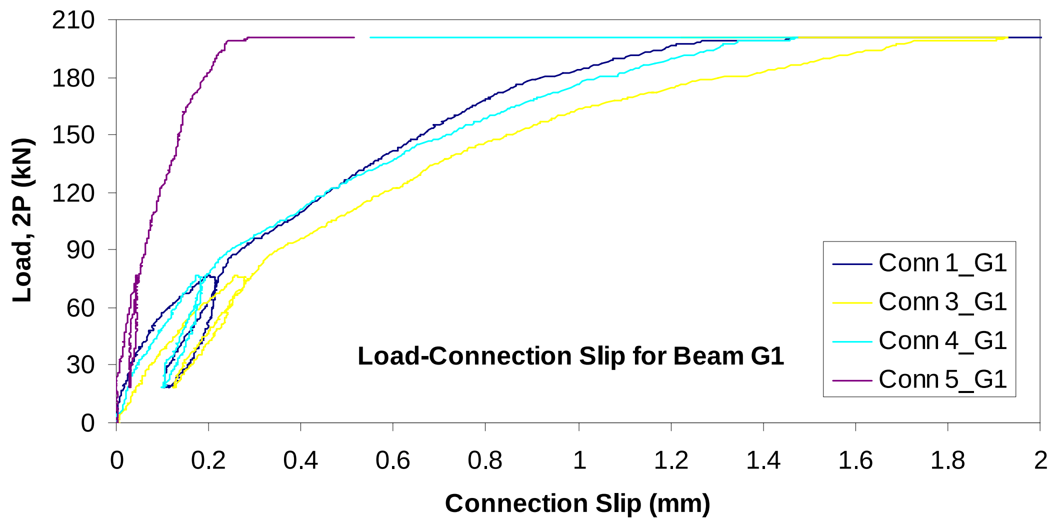

Load, 2P (kN)
120
100
80
60
40
20
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Connection Slip (mm)
Load-Connection Slip for Beam C2
Conn 1_C2
Conn 2_C2
Conn 3_C2
Conn 4_C2
Conn 5_C2


Load, 2P (kN)
90
80
70
60
50
40
30
20
10
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Connection Slip (mm)
Load-Connection Slip for Beam D1
Conn 1_D1
Conn 2_D1
Conn 3_D1


Load, 2P (kN)
100
80
60
40
20
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Connection Slip (mm)
Load-Connection Slip for Beam E1
Conn 1_E1
Conn 2_E1
Conn 3_E1


Load-Connection Slip for Beam E2
Load, 2P (kN)
Connection Slip (mm)
Conn 1_E2
Conn 2_E2
Conn 3_E2
0
10
20
30
40
50
60
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2


Load-Connection Slip for Beam F1
Load, 2P (kN)
Connection Slip (mm)
Conn 2_F1
Conn 3_F1
Conn 4_F1
0
20
40
60
80
100
120
140
160
180
0
1
2
3
4
5
6


Load-Connection Slip for Beam G1
Load, 2P (kN)
Connection Slip (mm)
Conn 1_G1
Conn 3_G1
Conn 4_G1
Conn 5_G1
0
30
60
90
120
150
180
210
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2

APPENDIX

8. Design, construction and setup of long-term push-out Test frames

This appendix describes in the design and construction of test frames used for the long-term push-out test.

8.1 General

3 types of connections were built and tested in the long-term with the objective to determine the creep coefficient of each connection for 1 year and then extended to 50 years: (1) Triangular notch 30°_60° 137l × 60d coach screw ϕ16 – T; (2) Rectangular notch 300l ×50d coach screw ϕ16 – R; and (3) Toothed metal plate 2×333l staggered – P. 3 frames were designed and built for this purpose. Plans for the test began in September 2007 with the design and drawings of the frames for the long-term push-out test while the construction of the frames started in middle January 2008. The frames were completed in late April 2008 and the long-term test commenced on the 19th May 2008. Another separate 3 specimens of each connection were setup without any load applied known as dummy specimens. These dummy specimens were used to monitor possible displacements induced by the self-weight of the specimens due to the change in temperature and relative humidity.

3 frames were constructed in the Structures Laboratory, Department of Civil and Natural Resources Engineering, University of Canterbury. The frames were assembled in a garage situated on Creyke Road located a few blocks from the University of Canterbury. This long-term push-out test on the connections shared the same garage with the already on-going long-term test on 3 timber-concrete composite beams. The garage is sheltered on all sides without any insulation on the walls and, under unheated and uncontrolled environmental condition. The stages involved in preparing for the frames are as follows: (1) design of long-term frames; (2) construction of frames; and (3) assembly of frames.

8.2 Design of long-term frames

Each frame has been designed for strength in bending, deflection, overall stability and capable to exert the required service load on the push-out specimen. The frame comprised of a bottom horizontal member and a single vertical member pivoted to the top horizontal member acting as a lever arm. The pins, bolts and welds were carefully designed and implemented due to the large amount of service loads imposed on the frames. Existing weights from other projects in the form of concrete blocks of 1000*b* × 1000*w* × 330*h* mm weighing 823 kg each or equivalent to 8.07 kN were used as loads in the frames. The service load imposed on each push-out specimen was defined as $0.3F_{max}$ representing the quasi-permanent part of the serviceability design load where F_{max} is the maximum strength of the connection determined from the short-term push-out test. The length (L_R) of each frame housing the 3 different connections were calculated based on the weight of the concrete block so that the force exerted on the push-out specimen was equivalent to $0.3F_{max}$. The point of load application at the specimen to the pivot was decided as 400 mm so that the specimen can be seated closest to the vertical member of the frame. Fig. A8-1 shows a typical set up of a long-term push-out test frame and Table A8-1 summarizes the design details for the frames. The top horizontal member of the frames were designed with universal column 200 UC 46.2 kg/m while the lower horizontal, vertical and other stabilizing members were designed from single or double parallel flange channels 200 PFC 22.9 kg/m.

Table A8-1. Design details for long-term push-out test frames

Frame	Fmax (kN)	0.3Fmax (kN)	Weight (kN)	LR (mm)
1	165	50	8.07	2680
2	240	72	8.07	3770
3	276	83	16.14	2260

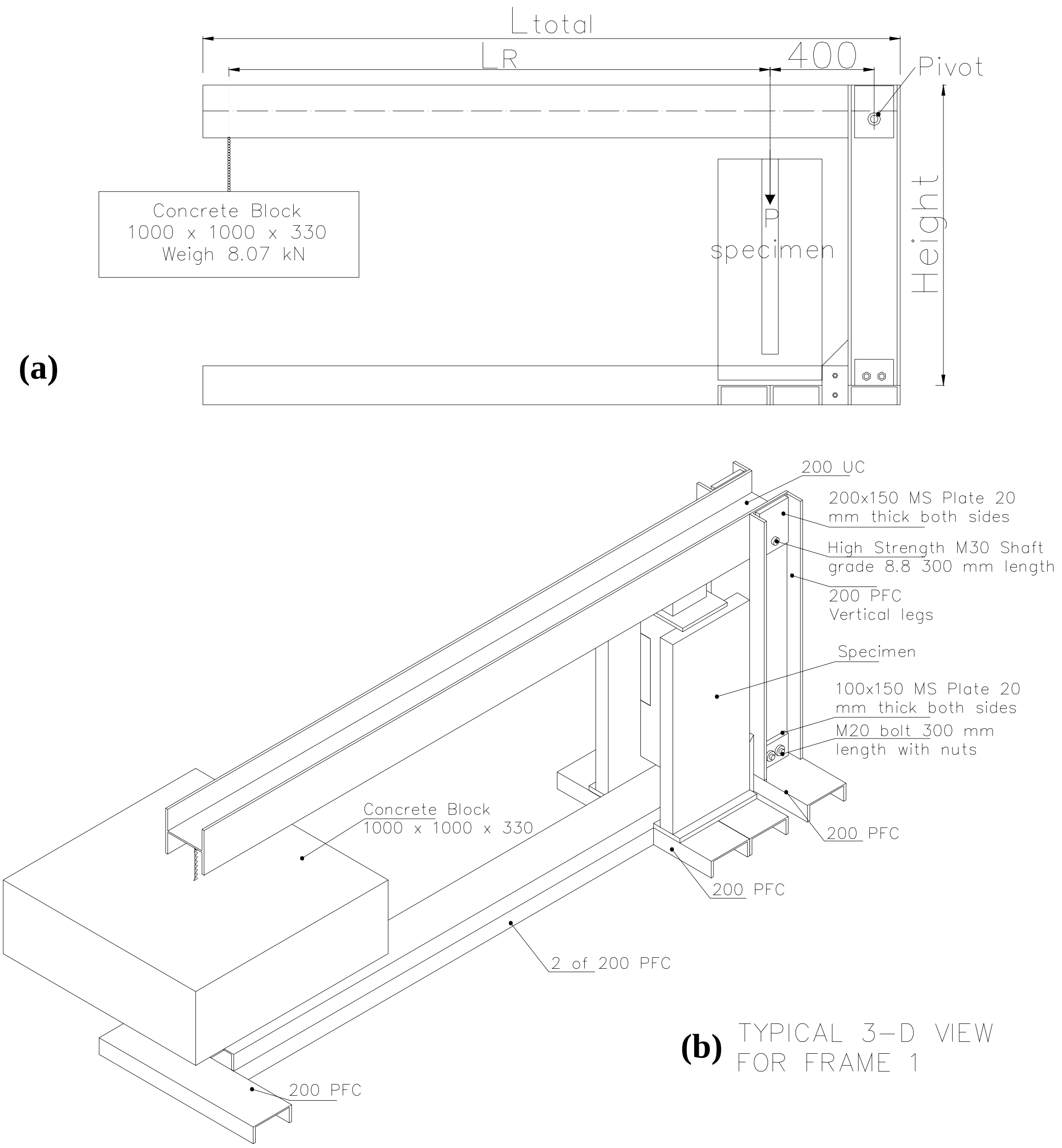


Fig. A8-1. Typical set up of a long-term push-out test frame (a) Side elevation; and (b) 3-dimensional view of frame

8.3 Construction of frames

The universal columns and parallel flange channels came in lengths of 6 m and had to be cut to the required length according to the drawings. Fig. A8-2 to Fig. A8-4 presents the side elevations and plan views for frames 1, 2 and 3 for triangular notched connection, rectangular notched connection and toothed metal plate connection push-out specimens respectively. Fig. A8-5 and Fig. A8-6 illustrate the connection details for the 3 frames. Construction of all frames took approximately 2 months by one technician working 8 hours a day and 5 days a week. Each frame was constructed in smaller elements so that they can be transported to the garage where the long-term test was housed for 1 year. The construction tasks were tedious and challenging as the sections were heavy and long (ranging from 2.2 to 3.8 m and 100 to 175 kg). Throughout the construction, there were limitations of space in the laboratory and availability of the gantry crane. All the elements of the frame were carefully coated with a layer of antirust paint, followed by a layer of blue paint.

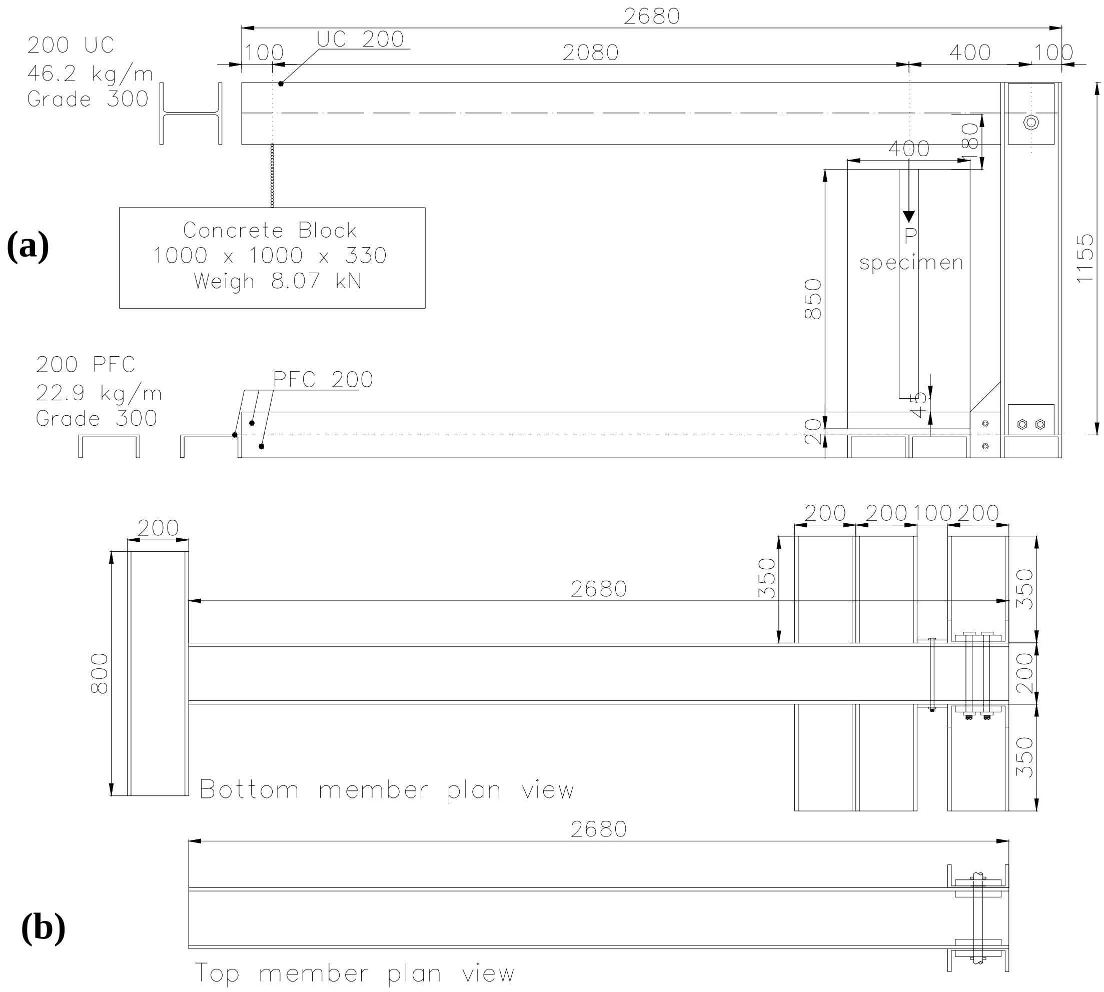


Fig. A8-2. Frame 1 for triangular notched connection push-out specimen: (a) Side elevation; and (b) Bottom and top member plan view

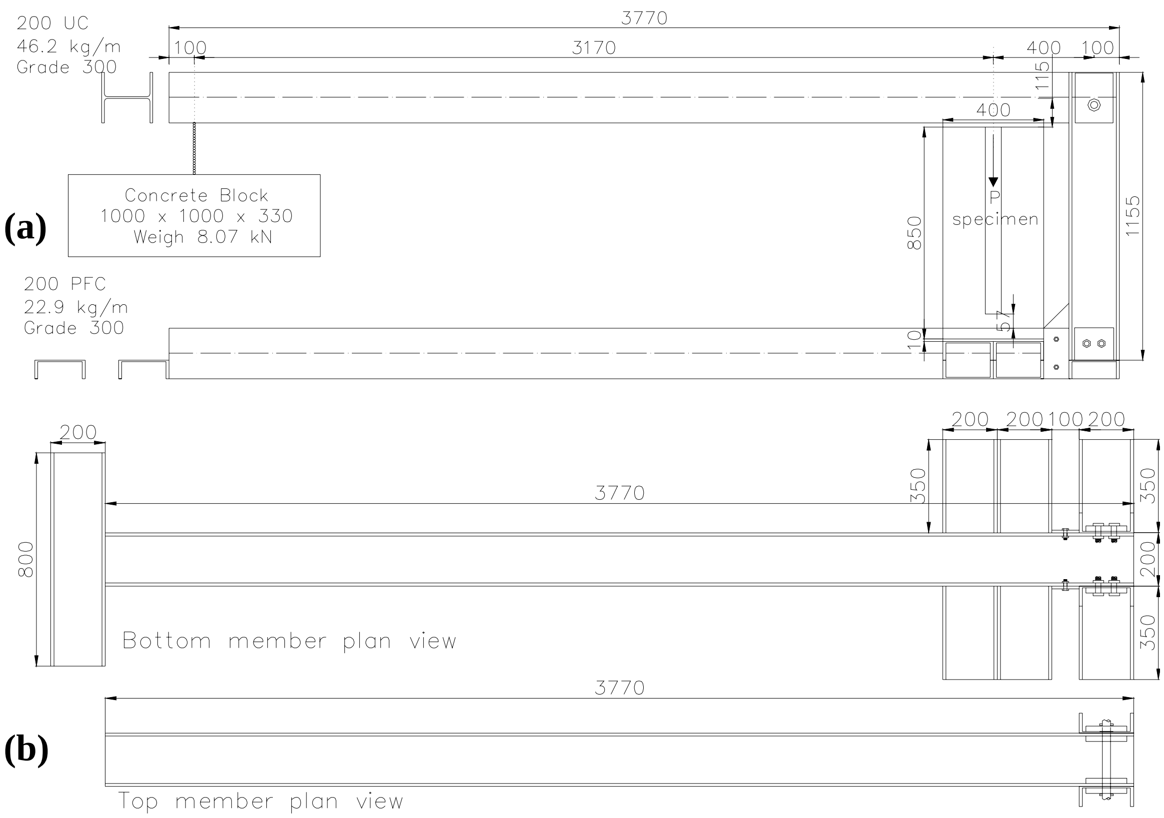


Fig. A8-3. Frame 2 for rectangular notched connection push-out specimen: (a) Side elevation; and (b) Bottom and top member plan view

8.4 Assembly of frames

The ready elements were transported to the garage and assembled there because of logistic and handling reasons. Due to the space and head room limitations in the garage, and the physical conditions of the elements, the assembly tasks have to be carefully thought and planned.

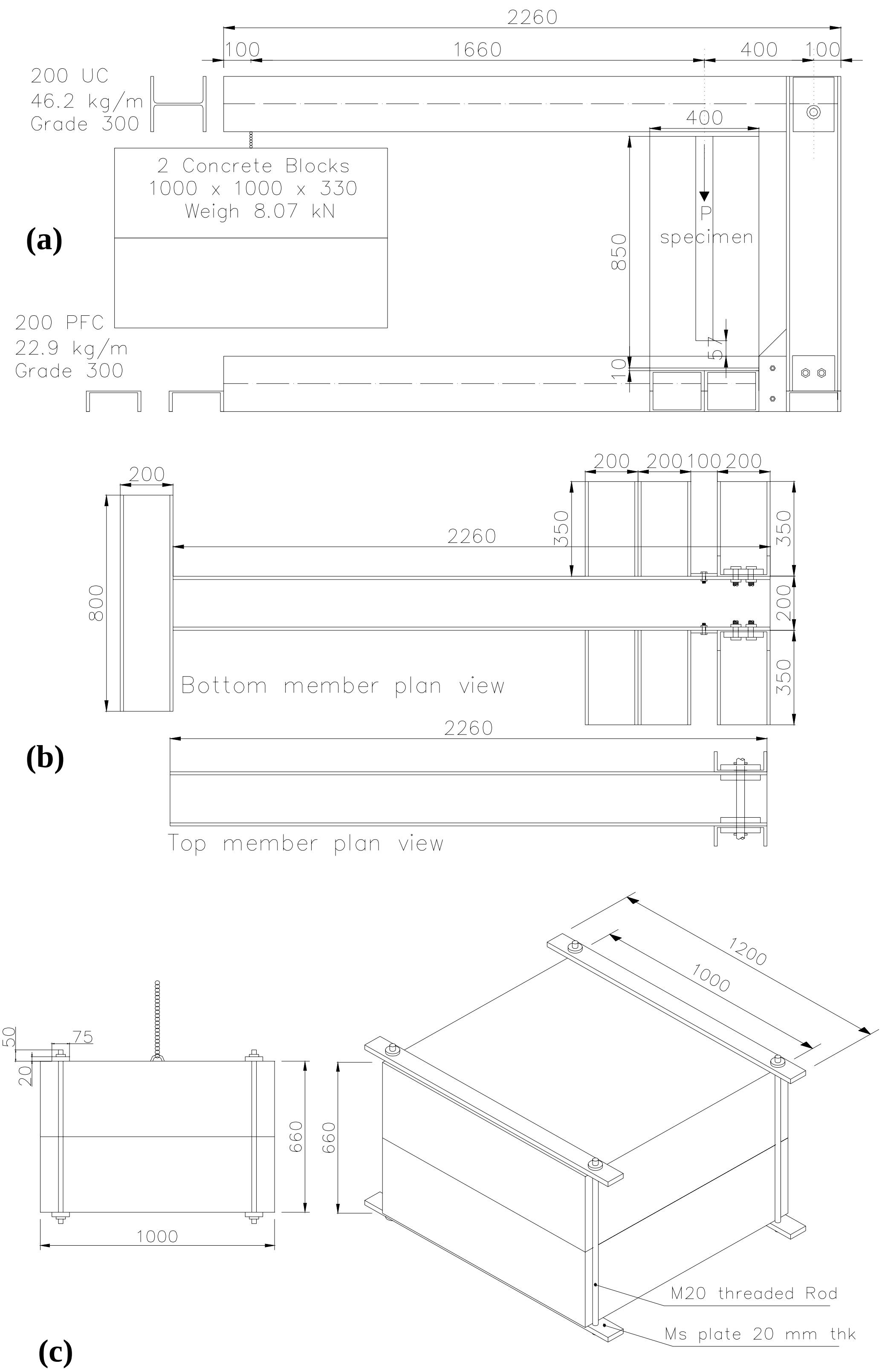


Fig. A8-4. Frame 3 for toothed metal plate connection push-out specimen: (a) Side elevation; (b) Bottom and top member plan view; and (c) Coupling of 2 concrete blocks together acting as weight in frame

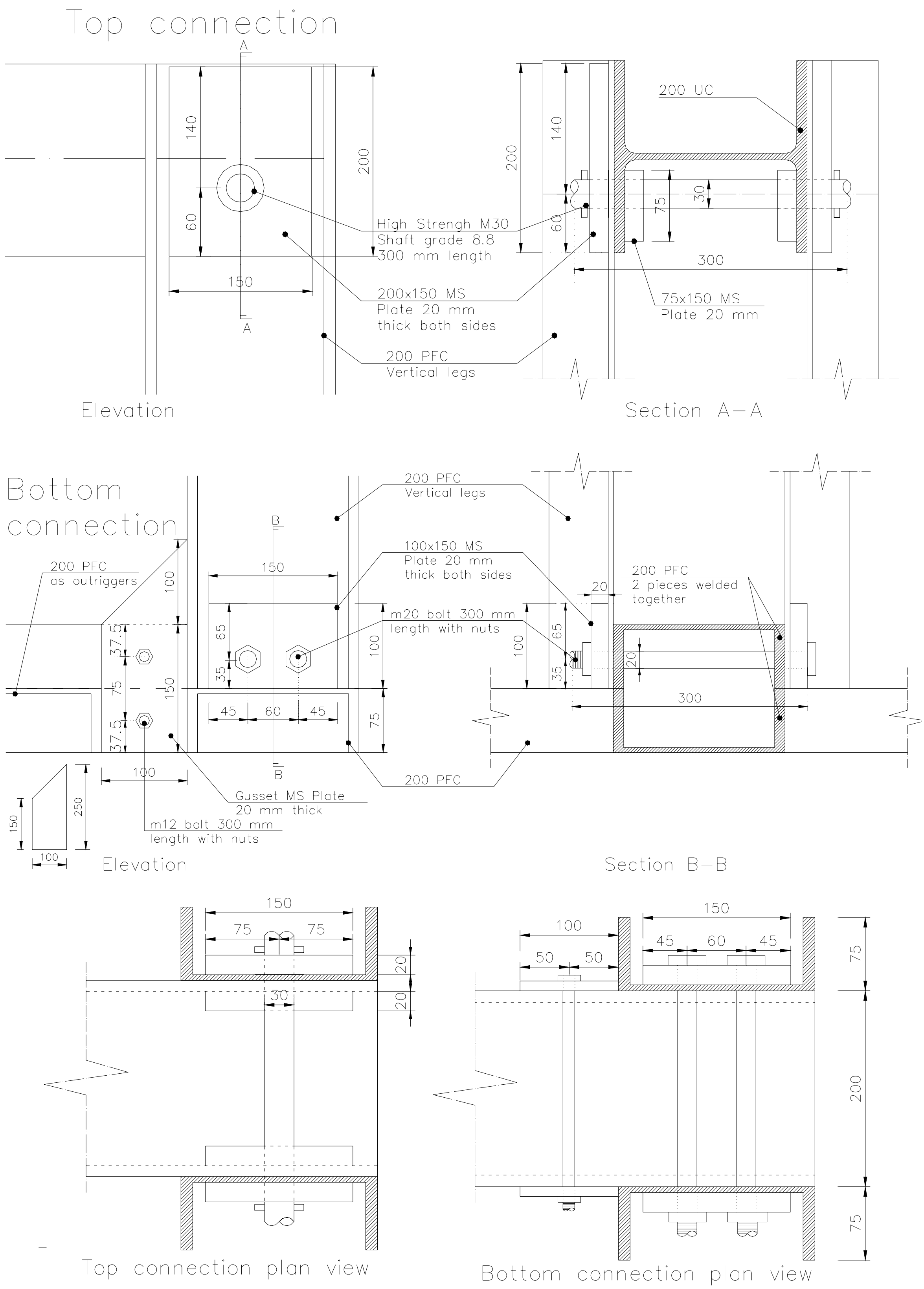


Fig. A8-5. Connection details for frame 1

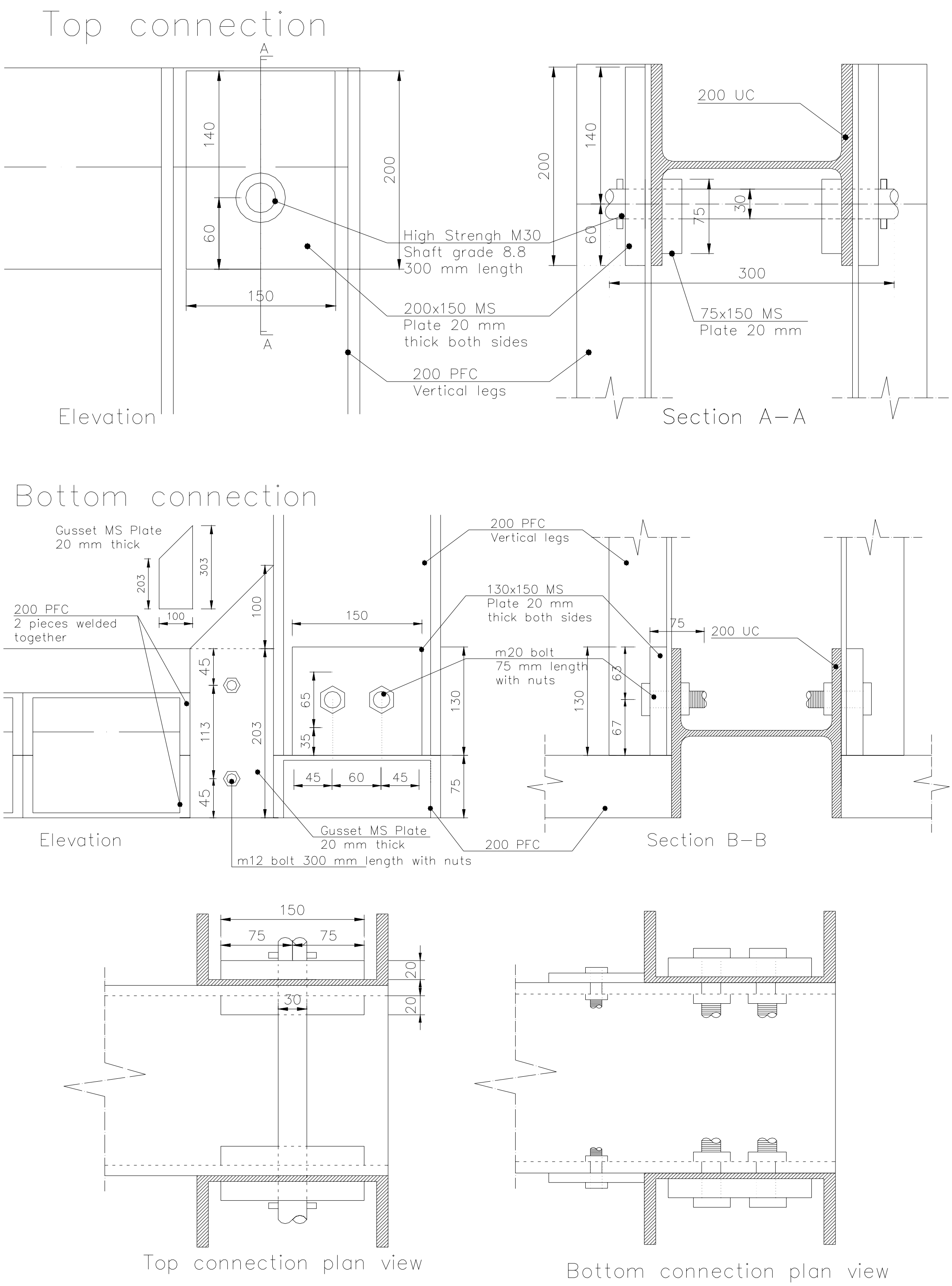


Fig. A8-6. Connection details for frames 2 and 3

Due to the floor and head room constraints in the garage, and the heavy weight of the specimens, pieces of loading frames and concrete weights, 2 special lifting frames: (1)

high frame with pulleys and chain block; and (2) low frame with rollers, have to be made for the purpose of assembling the test frames with the specimens under them. The lower frame was used to sit the concrete weight and pushed it into position under the test frame as shown in Fig. A8-7(a) and (b). The high lifting frame was used to lift the top member of the test frame before bolting it to the vertical member of the test frame (Fig. A8-7(c)). It was also used to lift the push-out specimen to position under the test frame as shown in Fig. A8-7(d).

Fig. A8-7. Two specially built lifting frames for use in the assembly of test frames in garage: (a) and (b) Lower frame to si the weights and pushed them to position; (c) and (d) High frame fixed with pulleys and chain block to lift top member of frame or push-out specimen to position

Fig. A8-8. Assembly of long-term push-out test frames: (a) Moving elements of a frame into the garage; (b) Positioning push-out specimen for frame 3; (c) Locating concrete weights for frame 3; (d) Frame 3 with push-out specimen and weights fully set up; (e) Hydraulic jack with LVL blocks used to temporarily held up frames before being loaded; and (f) Concrete weight fastened to top member of frame

The assembly of all frames with specimens in placed to be loaded took a total of 8 days in May 2008. The handling and maneuvering of each push-out specimen was very difficult and required at least two men due to the weight of the specimen being in the range of 100 to 150 kg and its odd irregular shape. The specimens had to be seated properly so as to avoid any unevenness or eccentricity. Several 10 and/or 20 mm steel plates were used as seats for this purpose and also to eliminate any height discrepancies. Similarly 20 mm steel plates were slotted in between the top member and the LVL member of the specimen where the load was applied onto. Individual LVL blocks or pieces were inserted to provide a uniformly distribution of the load. Fig. A8-8 shows the assembly process of the test frames in the garage.

Fig. A8-9. Potentiometers mounted to push-out specimens and loaded in test frames: (a) and (b) Frame 1 for triangular notched connection; and (c) and (d) Frame 2 for rectangular notched connection

8.5 Instrumentation for connection push-out test

Each push-out specimen under the test frame was mounted with 4 numbers (2 on each side) of 30 mm potentiometers adjacent to the connection (Fig. A8-9) while the dummy specimens mounted with 2 potentiometers each with the purpose to measure the relative slips in the connections. The positioning of the potentiometers was similar to that of the push-out specimens in the short-term test.

The setup of the loading frames and push-out specimens with all the instrumentation were ready on the 15th May 2008. A data acquisition trial run of the specimens under no loads was carried out for a few days in order to eliminate any unnecessary errors. Finally on the 20th May 2008, the hydraulic jacks supporting the concrete weights were released to apply the load on the specimens in the frames hence commencing the long-term test for a period of 1 year. Initial sampling rate was set at every 1 minute for the first two days followed by every 15 minutes in the next 1 week and subsequently every 1 hour. Temperature and relative humidity data were taken from the existing long-term test setup for LVL-concrete composite beams in the garage. The test was monitored on a weekly basis and the data backed up in order to avoid any possible loss of data.

APPENDIX

9. Construction and setup of long-term beam test

This appendix describes the construction and setting up of beams for long-term tests.

9.1 Construction and setup of long-term test beams – photographs

Three beams were constructed in a garage and tested in the long-term under sustained load. Fig. A9-1 to Fig. A9-3 show the construction and setting up of a 600 mm wide TCC single LVL with 300 mm rectangular notched coach screw connection and Fig. A9-4 to Fig. A9-7 for a 1200 mm wide TCC double LVL with metal plate connection.

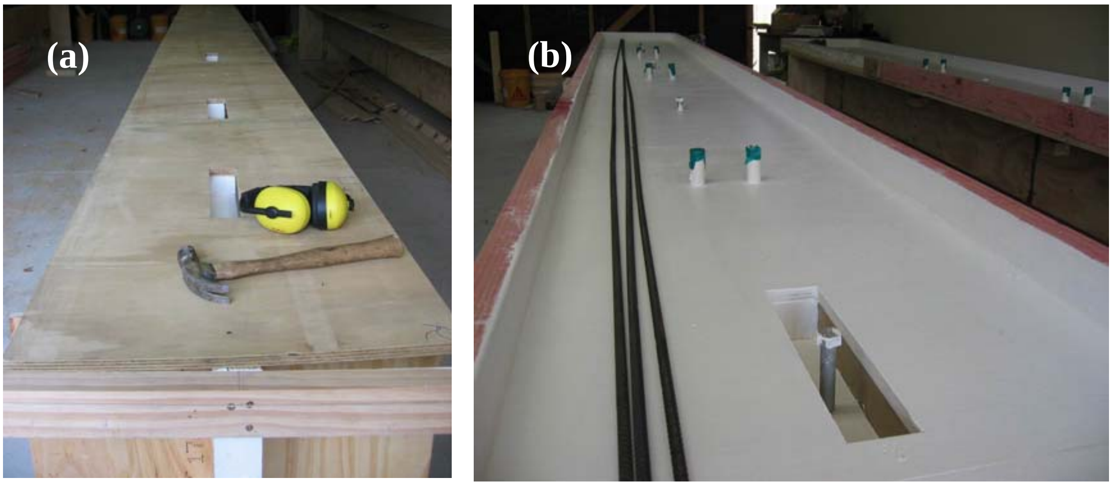


Fig. A9-1. (a) Construction of formwork for single LVL TCC 600 mm flange; (b) formwork ready with a coat of acrylic white paint and 300 mm rectangular notched with coach screw

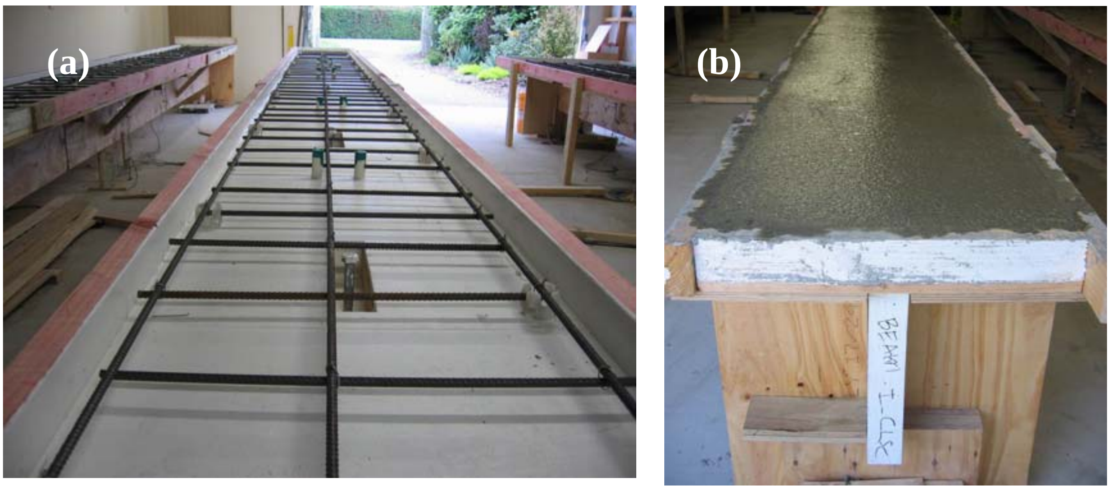


Fig. A9-2. (a) Reinforcement laid; (b) after concrete pour

Fig. A9-3. (a) TCC beam ready for loading; (b) water containers used as loads on beam

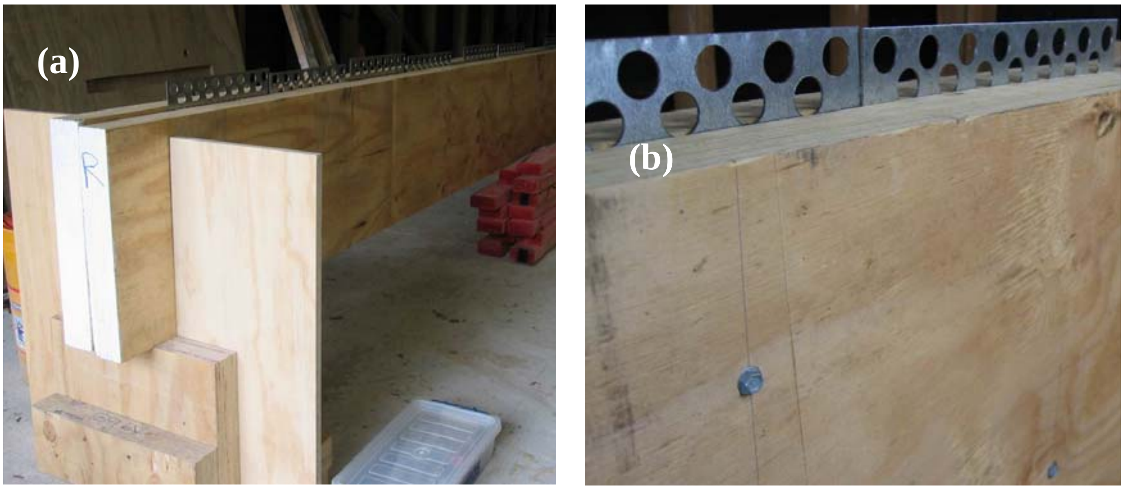


Fig. A9-4. (a) Setting of double LVL for construction of 1200 mm wide TCC; (b) 2-LVL connected using type 17 screw

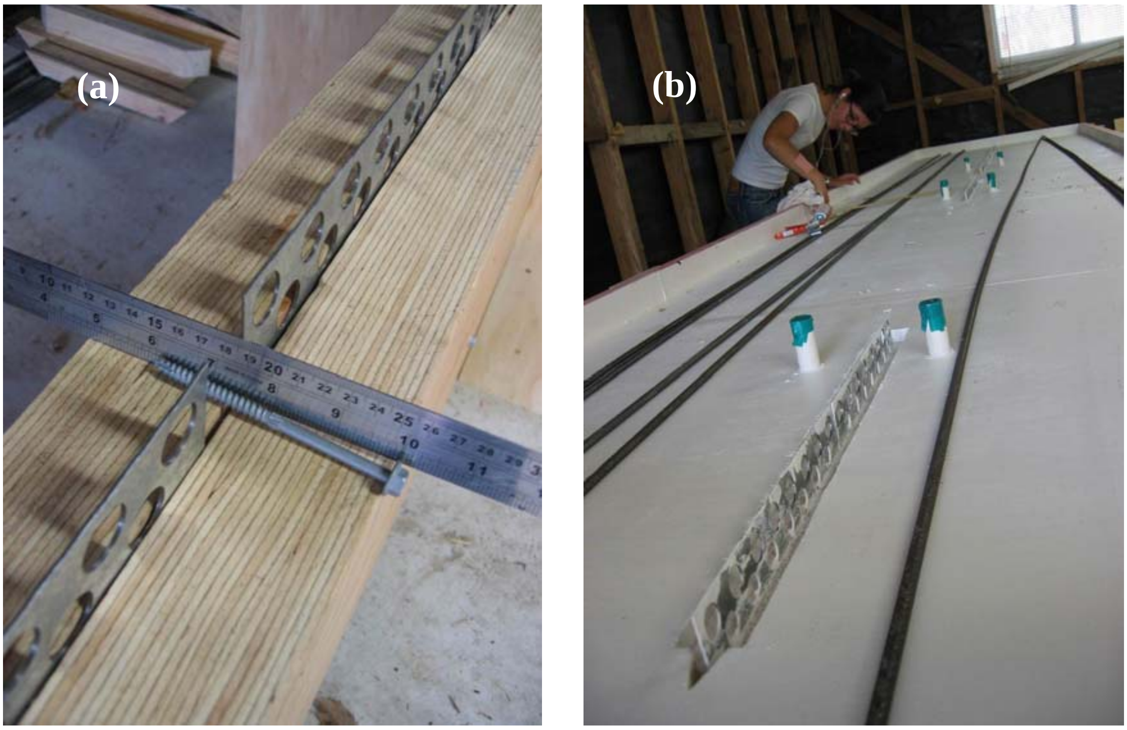


Fig. A9-5. (a) 100 mm long type 17 screw used to connect 2-LVL together; (b) preparation of formwork and metal plate connection along the length of beam

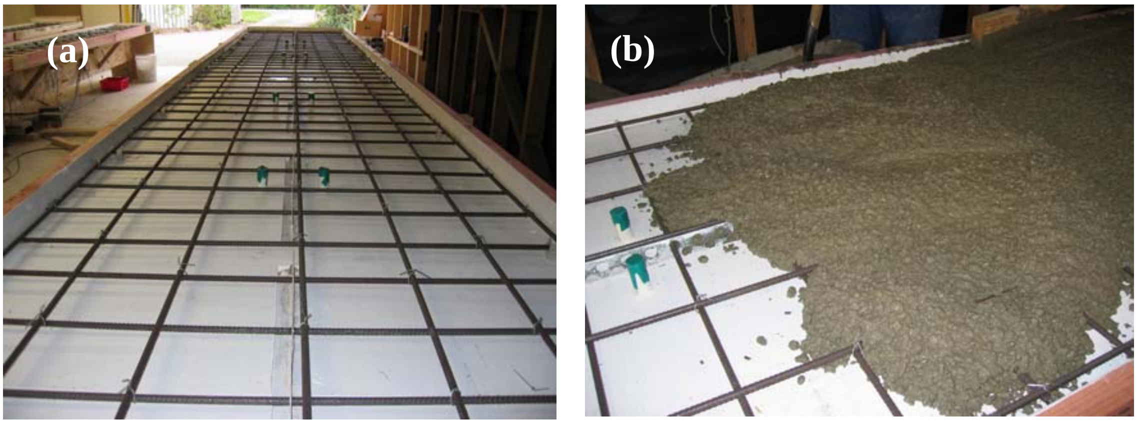


Fig. A9-6. (a) Reinforcement laid and ready for concrete; (b) concreting process

Fig. A9-7. (a) Completed concreting, surface slab preparation; (b) curing with Hessian sacks and covered with tarpaulin sheets

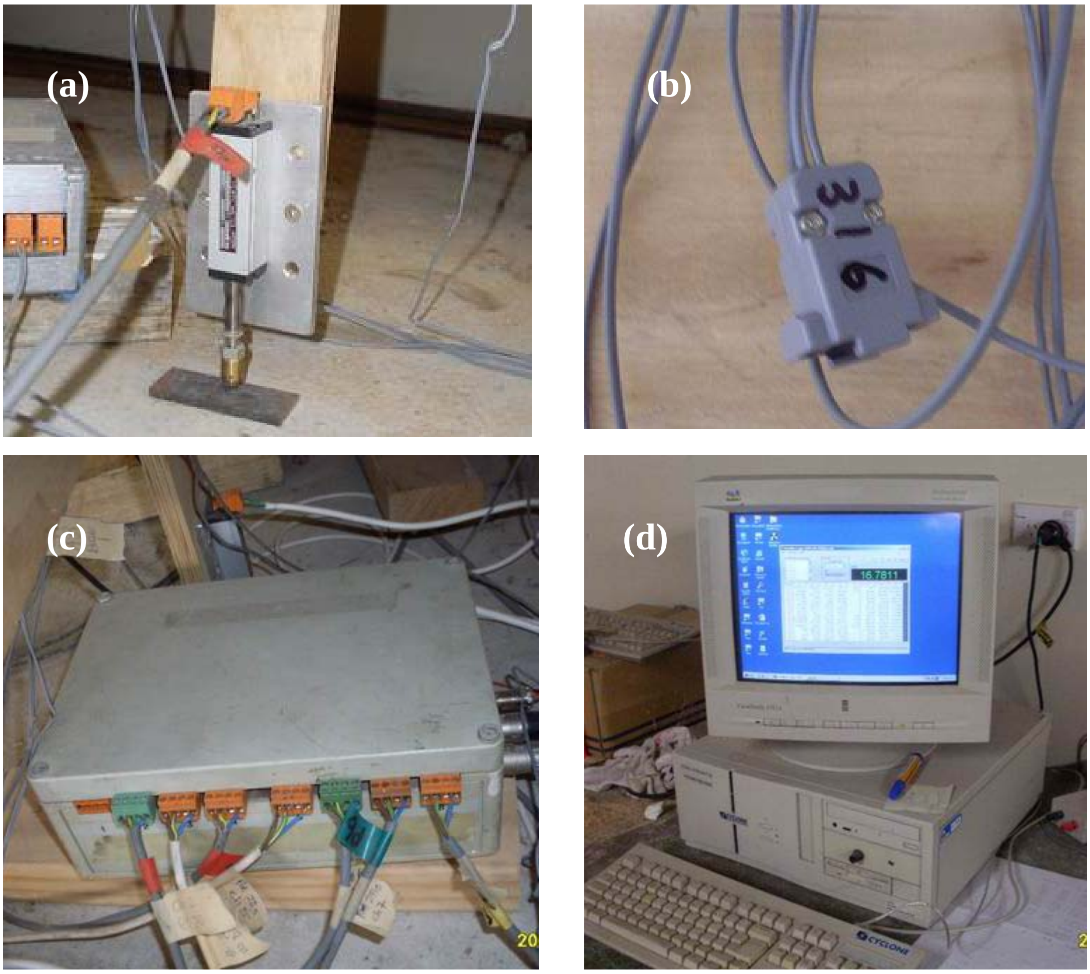


Fig. A9-8. (a) 30 mm potentiometer; (b) relative humidity and temperature sensors contained inside; (c) data acquisition box; and (d) computer recording long-term test automatically

9.2 Instrumentation for beam test

30 mm potentiometers with ± 0.7% accuracy were used to measure displacements at midspan and at the supports of the each beam (Fig. A9-8(a)). Relative humidity and temperature were measured using HIH-4000 Series humidity sensors and LM-35 temperature sensors, respectively (Fig. A9-8(b)). The sampling rate was every minute during the concreting process and for the first 24 hours and after, every hour for the remainder of the long-term test duration. All the potentiometers and sensors were calibrated, assigned a channel each in a data acquisition box (Fig. A9-8(c)) and connected to a computer which recorded all the measurements automatically (Fig. A9-8(d)).

APPENDIX

10. Design span tables

This appendix presents span tables for semi-prefabricated LVL-concrete composite floors. The span tables give the safe live load in kN/m^2 for M-section module 2400 mm total width (Fig. A10-1) with 3 connection types (Fig. A10-2): (1) 300 mm long rectangular notch cut in the LVL joist and reinforced with a 16 mm diameter coach screw (R-300); (2) triangular notch reinforced with the same coach screw (T); and (3) two 333 mm long toothed metal plates (P).

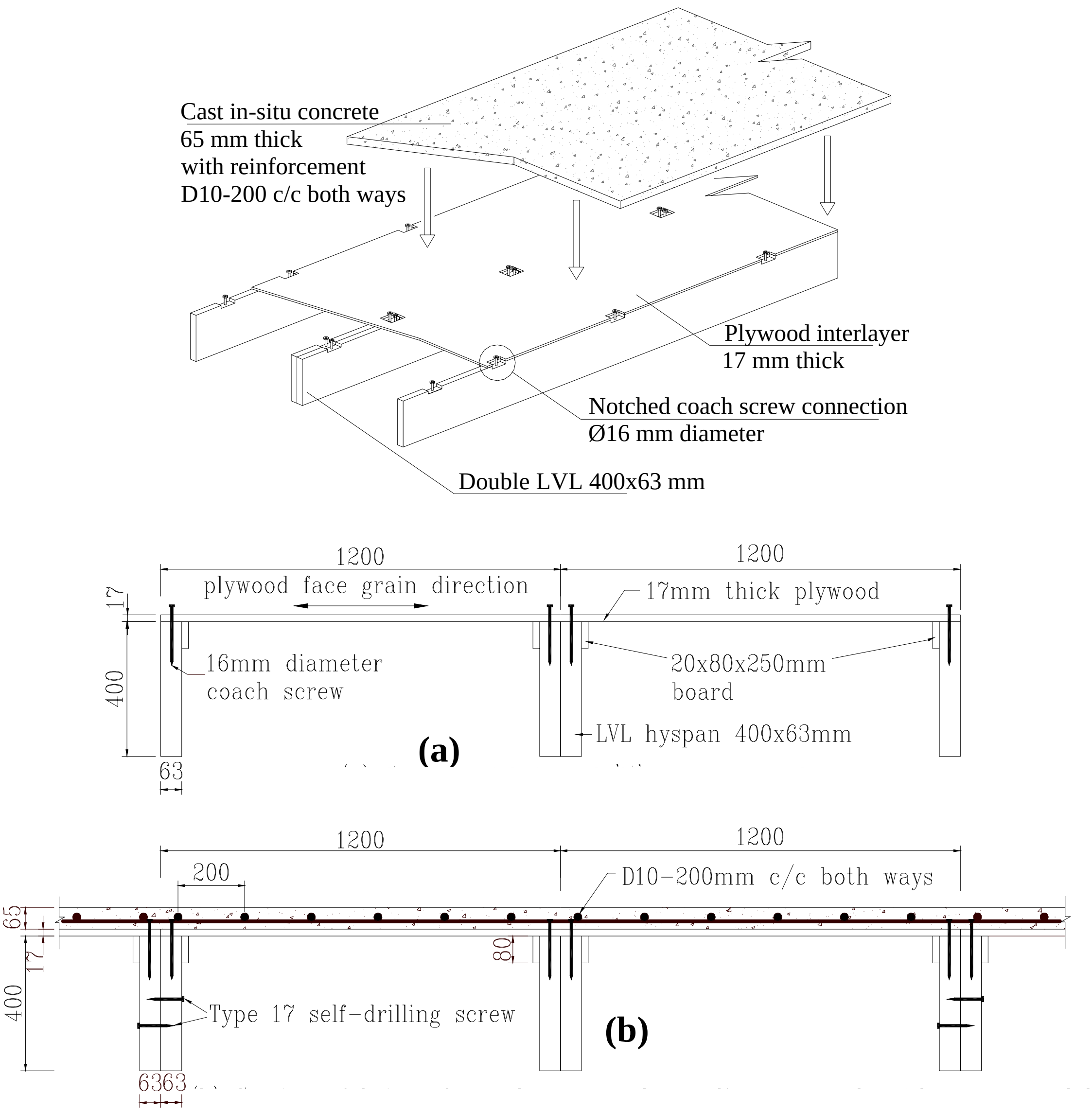


Fig. A10-1. Semi-prefabricated LVL-concrete composite floor system (top): (a) Module M 2.4 m; and (b) Single module connected to adjacent modules using type 17 screw and concrete cast on top

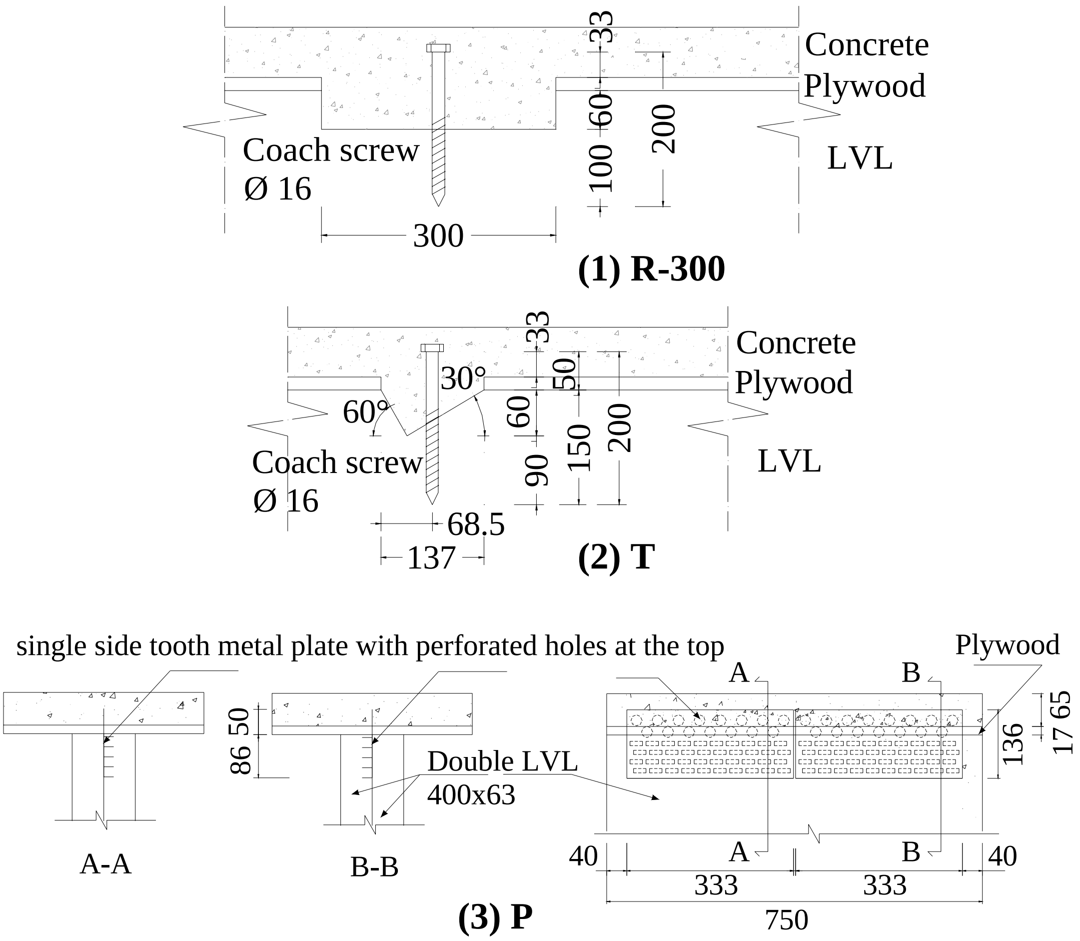


Fig. A10-2. Three types of connection: (1) 300 mm long rectangular notch cut in the LVL joist and reinforced with a 16 mm diameter coach screw (R-300); (2) triangular notch reinforced with the same coach screw (T); and (3) two 333 mm long toothed metal plates (P) (dimensions in mm)

10.1 Module M 2.4 m Connection R-300

Module **M 2.4m**

Connection **R-300**

Module	Safe Live Load (kN/m^2) with number of connections (#) along span													
	Span (m)													
	6	#	7	#	8	#	9	#	10	#	12	#	15	#
300LVL	3.6	4	4.6	6	3.7	8								
400LVL					4.6	6	3.3	6	3.6	8				
600LVL							6.0	6	5.6	8	3.8	8	2.2	12

Minimum support bearing length is 63 mm

Minimum support bearing length is 75 mm

Notes:

1. LVL is Truform recipe with E = 10.7 GPa. Each module unit consist of double LVL b = 2 x 63 mm in the middle and single LVL b = 63 mm on the outer edges.
2. Concrete is 65 mm thick Grade 35 with low shrinkage of maximum 650 microstrain drying shrinkage at 28 days.
3. Reinforcement both ways in concrete flange required for cracking control.
4. Stiffness and strength of connection and its long-term creep coefficient were obtained experimentally.
5. Satisfies all verifications in the short- and long-term ULS and SLS using the γ-method according to Annex B of Eurocode 5.
6. Modification factors for LVL are according to NZ3603: 1993 and LVL Specific Design Information (CHH); and for concrete, NZ3101: Part 1: 2006.
7. Medium duration of load factor for strength, k_1 = 0.8 has been assumed. Duration of load factor for deflection, k_2 = 2.0 has been assumed.
8. Total dead load, G includes Superimposed dead load 1 kN/m^2 and self-weight 2 kN/m^2.
9. Load combinations:
   ULS, 1.2G + 1.5Q, and 1.2G + 1.5P where P = 2.7 kN.
   SLS long-term, G + 0.4Q
10. Deflection limit in the short-term only for live load is span/300 and long-term is span/200.
11. Vibration has been checked for 1 kN point load at mid-span with deflection less than 1 mm.
12. Effect of inelastic strains in the concrete has not been considered. This may cause excess deflection in the long-term. The amount of this deflection depends on the environmental condition, type of concrete and its thickness, and type of connection.

Module **M 2.4m**
Connection **R-300**

Span-Module	Connection total number	Connection position (m) from end of beam to centre of connection for half span					
		1st	2nd	3rd	4th	5th	6th
6-300LVL	4	0.500	1.586	-	-	-	-
7-300LVL	6	0.450	1.061	2.153	-	-	-
8-300LVL	8	0.400	0.854	1.586	2.667	-	-
8-400LVL	6	0.450	1.212	2.460	-	-	-
9-400/600LVL	6	0.450	1.364	2.768	-	-	-
10-400/600LVL	8	0.400	1.067	1.982	3.333	-	-
12-600LVL	8	0.425	1.281	2.379	4.000	-	-
15-600LVL	12	0.400	1.015	1.786	2.683	3.804	5.459

10.2 Module M 2.4 m Connection T

Module **M 2.4m**
Connection **T**

Module	Safe Live Load (kN/m^2) with number of connections (#) along span													
	Span (m)													
	6	#	7	#	8	#	9	#	10	#	12	#	15	#
300LVL	1.6	4	2.4	6	2.3	8								
	4.2	6	3.8	8	3.0	10								
400LVL					2.6	6	2.3	8	2.2	12				
					3.6	8	3.1	10						
600LVL							3.0	6	3.2	8	3.0	14	1.6	16
							4.6	8						

Minimum support bearing length is 63 mm

Notes:

1. LVL is Truform recipe with E = 10.7 GPa. Each module unit consist of double LVL b = 2 x 63 mm in the middle and single LVL b = 63 mm on the outer edges.
2. Concrete is 65 mm thick Grade 35 with low shrinkage of maximum 650 microstrain drying shrinkage at 28 days.
3. Reinforcement both ways in concrete flange required for cracking control.
4. Stiffness and strength of connection and its long-term creep coefficient were obtained experimentally.
5. Satisfies all verifications in the short- and long-term ULS and SLS using the γ-method according to Annex B of Eurocode 5.
6. Modification factors for LVL are according to NZ3603: 1993 and LVL Specific Design Information (CHH); and for concrete, NZ3101: Part 1: 2006.
7. Medium duration of load factor for strength, k_1 = 0.8 has been assumed. Duration of load factor for deflection, k_2 = 2.0 has been assumed.
8. Total dead load, G includes Superimposed dead load 1 kN/m^2 and self-weight 2 kN/m^2.
9. Load combinations:
   ULS, 1.2G + 1.5Q, and 1.2G + 1.5P where P = 2.7 kN.
   SLS long-term, G + 0.4Q
10. Deflection limit in the short-term only for live load is span/300 and long-term is span/200.
11. Vibration has been checked for 1 kN point load at mid-span with deflection less than 1 mm.
12. Effect of inelastic strains in the concrete has not been considered. This may cause excess deflection in the long-term. The amount of this deflection depends on the environmental condition, type of concrete and its thickness, and type of connection.

Module **M 2.4m**
Connection **T**

Span-Module	Connection total number	Connection position (m) from end of beam to centre of connection for half span							
		1st	2nd	3rd	4th	5th	6th	7th	8th
6-300LVL	4	0.440	1.590	-	-	-	-	-	-
	6	0.260	0.910	1.850	-	-	-	-	-
7-300LVL	6	0.250	0.750	1.390	-	-	-	-	-
	8	0.250	0.750	1.390	2.330	-	-	-	-
8-300LVL	8	0.270	0.850	1.590	2.670	-	-	-	-
	10	0.200	0.660	1.250	1.900	2.800	-	-	-
8-400LVL	6	0.270	0.850	1.590	-	-	-	-	-
	8	0.270	0.850	1.590	2.670	-	-	-	-
9-400LVL	8	0.300	0.960	1.785	3.000	-	-	-	-
	10	0.240	0.745	1.400	2.200	3.200	-	-	-
9-600LVL	6	0.415	1.360	2.770	-	-	-	-	-
	8	0.300	0.960	1.785	3.000	-	-	-	-
10-400LVL	12	0.220	0.680	1.190	1.790	2.540	3.640	-	-
10-600LVL	8	0.335	1.070	1.980	3.330	-	-	-	-
12-600-LVL	14	0.260	0.810	1.430	2.100	2.800	3.550	4.400	-
15-600-LVL	16	0.260	0.650	1.300	2.050	2.850	3.800	4.800	5.900

10.3 Module M 2.4 m Connection P

Module **M 2.4m**
Connection **P**

Module	Safe Live Load (kN/m^2) with number of connections (#) along span													
	Span (m)													
	6	#	7	#	8	#	9	#	10	#	12	#	15	#
300LVL	0.7	4	1.0	6	0.9	8								
400LVL					1.7	8	1.3	10	0.9	12				
600LVL							3.0	10	2.4	12	1.6	14	0.7	18

Minimum support bearing length is 63 mm

Notes:

1. LVL is Truform recipe with E = 10.7 GPa. Each module unit consist of double LVL b = 2 x 63 mm in the middle and single LVL b = 63 mm on the outer edges.
2. Concrete is 65 mm thick Grade 35 with low shrinkage of maximum 650 microstrain drying shrinkage at 28 days.
3. Reinforcement both ways in concrete flange required for cracking control.
4. Stiffness and strength of connection and its long-term creep coefficient were obtained experimentally.
5. Satisfies all verifications in the short- and long-term ULS and SLS using the γ-method according to Annex B of Eurocode 5.
6. Modification factors for LVL are according to NZ3603: 1993 and LVL Specific Design Information (CHH); and for concrete, NZ3101: Part 1: 2006.
7. Medium duration of load factor for strength, k_1 = 0.8 has been assumed. Duration of load factor for deflection, k_2 = 2.0 has been assumed.
8. Total dead load, G includes Superimposed dead load 1 kN/m^2 and self-weight 2 kN/m^2.
9. Load combinations:
   ULS, 1.2G + 1.5Q, and 1.2G + 1.5P where P = 2.7 kN.
   SLS long-term, G + 0.4Q
10. Deflection limit in the short-term only for live load is span/300 and long-term is span/200.
11. Vibration has been checked for 1 kN point load at mid-span with deflection less than 1 mm.
12. Effect of inelastic strains in the concrete has not been considered. This may cause excess deflection in the long-term. The amount of this deflection depends on the environmental condition, type of concrete and its thickness, and type of connection.

Module **M 2.4m**
Connection **P**

Span-Module	Connection total number	Connection position (m) from end of beam to centre of connection for half span								
		1st	2nd	3rd	4th	5th	6th	7th	8th	9th
6-300LVL	4	0.500	1.586	-	-	-	-	-	-	-
7-300LVL	6	0.450	1.150	2.153	-	-	-	-	-	-
8-300/400LVL	8	0.375	1.050	1.750	2.667	-	-	-	-	-
9-400/600LVL	10	0.375	1.050	1.720	2.390	3.160	-	-	-	-
10-400/600LVL	12	0.375	1.050	1.730	2.410	3.090	3.770	-	-	-
12-600LVL	14	0.375	1.050	1.725	2.400	3.075	3.750	4.450	-	-
15-600LVL	18	0.375	1.050	1.725	2.400	3.075	3.750	4.425	5.100	5.775